Engineering Record

A Weekly Journal Devoted to Civil Engineering and Contracting

PUBLISHED BY McGRAW PUBLISHING COMPANY, INC.

Volume 69 NEW YORK, SATURDAY, MAY 2, 1914 Number 18

Cape Cod Canal

WITH simple ceremony the final connection of the waters of Cape Cod and Buzzard's bays was made last week. To those not conversant with the conditions the building of a canal only 8 miles long seems a matter of little importance. As a matter of fact, while not a stupendous engineering feat, the new waterway has an importance due to its location far beyond its position as an engineering structure. The waters outside Cape Cod are notably turbulent, a matter of far more moment in the days of the sailing vessel than now, but even of sufficient importance at present to lead the promoters of the canal to invest a large sum in the enterprise. The added safety, freedom from delays and shortened distance between Boston and New York, and intermediate points, are counted on to give the canal ample business. The culmination of the enterprise is locally of very great interest, for the feasibility of the canal was long realized and its need urged for fully two and a half centuries. Its practical completion, therefore, is an event of greater importance than the simple ceremonies of last week would seem to indicate.

Valuation and Capitalization

RAILROAD authorities have all along maintained that a fair valuation would show that the railroads were under-capitalized and that those who were clamoring for valuation as a basis for rate reductions were going to be woefully disappointed. This contention is now impressively confirmed by the valuation of the Lehigh Valley Railroad under the direction of W. J. Wilgus, formerly vice-president and chief engineer of the New York Central. Mr. Wilgus's reputation no less than the care with which this valuation has been carried out will satisfy any reasonable mind that the Lehigh Valley figures are as correct as they can be made. They show that to reproduce the Lehigh Valley would cost $324,478,300, while the stock and bonds of this railroad aggregate only $184,000,000. The cost of reproduction is made up of these three principal items: right of way, real estate, buildings and shops, $228,499,050; equipment, $86,979,250; working capital and stores, $9,000,000. In making up these figures the Interstate Commerce Commission's system of classification of accounts was followed. If there had ever been any rime or reason in the notion that rates could be determined by capitalization and lowered in cases where overcapitalization was found, it would, conversely, now be in order for the Lehigh Valley to ask that it be allowed to charge rates which would earn a fair return on $140,000,000 more than its capitalization. But of course the argument is not supposed to work that way.

Simplification of Water Laws

RIPARIAN rights are a particularly opportune subject for discussion at present, when, due to recent exhaustive papers and the appointment of a special committee on a National water law by the American Society of Civil Engineers, more attention than ever before is being given to the legal aspects of water rights. The fundamental point to be settled in any consideration of a National statute is not any single detail or group of details of riparian law, but rather whether the riparian or the reasonable-use doctrine shall prevail. Without going into this important question here, it is not unreasonable to expect that no matter what the findings of the special committee above referred to, the doctrine of riparian rights will for a long time, and probably always, be with us. Consequently discussion of the definitions advanced by Mr. Horton in the article on page 495 and of the Tennessee decisions, which he cites, are well worth while. Further emphasis of his main point—that there is great need for more simple and more logical precedents regarding water rights—is hardly necessary, for that need has long been felt by all hydraulic engineers. Changes naturally must come slowly in the New England and the Eastern States, but those in which water is only now becoming of great importance have an opportunity to escape the entanglements which have developed in the older communities.

Limiting Industrial Districts

NOW that the Board of Estimate and Apportionment of New York City has been given power not only to restrict the height of buildings but also to district the city and to say what industries shall be allowed in each district, we may expect to have the legality of the extension of the police power to the latter subject thoroughly threshed out. If restriction of building heights is a legitimate exercise of the police function, regarding which there is apparently little doubt, it does seem likely that the other measure is also within proper bounds. In Germany this right of restriction is firmly established, while several American cities, though in a more limited degree than seems to be contemplated at New York, have restricted areas. The Mayor of New York openly expresses his doubt as to the constitutionality of the measure, though the president of the Board of Aldermen is positive that the measure cannot be successfully assailed. Nevertheless, as the Brooklyn "Eagle" puts it, "The constitutional question is one to delight lawyers." Evidently there is danger of confiscation. No right-minded person wants that, except when lack of restriction works damage to whole districts. Then the general good becomes paramount. That is the view the Engineering Record takes. The ultimate good of all our citizens in urban communities will be served by restriction, and it is hoped, therefore, that the New York law may stand the severest tests to which it can be put. The text of the new law is published on page 498 of this issue.

Local Railways in Canada

IT is gratifying to note the attitude which the acting Minister of Railways in Canada, J. D. Reid, holds toward the proposed "All-Red Railroad." The promoters of this new scheme sought to plan another Canadian transcontinental line, which would begin on the Labrador coast and end at the head of the Portland canal, but the project has met with so little favor at the hands of the railway committee of the House of Commons, that it is probable that the entire matter will be set aside indefinitely. Mr. Reid is reported to have intimated that Canada can get along very well for the present with the three transcontinental lines which have already been built. This may arouse some criticism in Canada, where the great need of railways has been—and, in many sections, still is—the greatest check to the rapid development of the country. However, it should be remembered that the interests of no small proportion of the western provinces will be closely identified with the success of the Grand Trunk Pacific and the Canadian Northern Railways. The same fair policy, which has fostered a certain degree of competition and has given Government aid to the new roads, should protect them now that an adequate number of trunk lines exist, and should assist them, insofar as it is within the province of the Government to do so, in developing the territory just being opened up. The country needs more railroad mileage, it is true, but the construction of branch lines or local railways which will serve as feeders to the present trunk roads is now the logical step in railroad extension in Western Canada. If Government aid is needed at all it will be for the branch lines that will soon be required in the northern areas of Saskatchewan, Alberta and British Columbia. It is a wise Government policy that neither hinders development by ultra-conservatism nor encourages capital in ill-advised investments.

Regulation of Competitions

ELSEWHERE in this issue is an account of the discussion before the American Institute of Consulting Engineers on the regulation of competitions. Because of their long experience with this matter and the progress they have made, it was quite natural to call upon the architects for their experience. The main features of their code—that of the American Institute of Architects—can be briefly summarized.

First, and most important, no member of the institute may take part in a competition unless it has been approved. It is evident that if that rule can be enforced, the rest, so far as the institute is concerned, is fairly simple, for it can refuse to approve competitions which are not drawn up in accordance with its rules. The most important of these rules are: First, that the plan of the competition shall be drawn up by a competent architect, thus assuring that all competitors will get the same information and that the terms are fair; second, that the judges shall be qualified architects, and third, that the winner shall be awarded a contract for the work. The last provision necessarily requires that no one shall be allowed to compete unless the owner is willing to give him the contract if he wins.

While these fundamentals are unusually simple, they are the result of long experience, involving many changes. The sole object is to insure equal treatment of all competitors, competent judges, and the execution of a contract with the successful competitor. If recent bridge competitions, that have attracted widespread attention, had been held under such rules, the severe criticism to which they have been subjected would not have been incurred.

There is certainty that qualified engineers will endorse heartily the rules worked out by the architects. It is manifest, however, that their adoption by the consulting engineers' institute will not bring about the millennium with any great speed. That is not a good reason, however, for hesitating, and while a code governing competitions adopted by the American Institute of Consulting Engineers would be mandatory on only about seventy-five men, the influence of that body is such that the effect would be felt throughout the profession.

It is to be hoped, therefore, that the committee on professional practice and ethics of the institute will report a code to that body. It will undoubtedly be modified, as compared with the code of the architects, largely because the institute has not the Nation-wide machinery possessed by the other body. No matter how elementary its form, however, it should have a tendency to shame those who have indulged in some of the recent profitless scrambles.

A Presentment on Consulting Engineers

AS noted in this journal last week, a New York grand jury, in a formal presentment, has criticised the consulting engineers of the Board of Water Supply. "The employment," the presentment said, "of a staff of salaried consulting engineers can only be justified upon the theory that these consulting engineers are such in fact and that when called upon for the expression of an opinion in connection with any contemplated work such opinion, when expressed, is their own independent opinion, reached by them without foreknowledge of the opinion already formed and expressed by the regular engineering staff of the said Board of Water Supply."

This quotation indicates a lack of full information concerning the methods of the organization criticised. Instead of submitting to the consulting engineers opinions, reports or plans in a final form for their first consideration, the chief engineer, directly or through other members of "the regular engineering staff of the Board," has habitually sought advice in the initial stages. The constant endeavor has been to focus early upon each problem the best experience, skill and judgment available in this carefully selected force. As the various parts of the work were developed, the consulting engineers were conversant with them step by step. Differences of opinion frequently arose, but usually additional information or further study secured agreement, whether the questions at issue were those of design, or construction, or administrative policy. In the few cases in which all the consulting engineers and the chief engineer could not reach an agreement, final decision necessarily lay with the chief engineer. If one of the consultants had had greater experience in the field of the problem under discussion, naturally his opinion was given the greater weight.

As the purpose was to construct a waterworks system rapidly and economically and not to conduct a debating society, co-operation rather than criticism was the dominant note. A problem having been brought to solution, the statement or report was drafted sometimes by the chief engineer, sometimes by one or more of the consulting engineers, and sometimes by a subordinate; before final writing, it was criticised by each one concerned. Sometimes separate reports were written. It is not believed that such "foreknowledge" and harmony sacrificed any useful degree of "independence" in opinions or militated against the candor and integrity of the parties giving such opinions.

Since these consulting engineers were men of sagacity and broad contact with the world, they were not infrequently called upon to advise the commissioners directly on questions of administrative policy as well as on engineering matters. Employment on a project of such magnitude as the Catskill water system of a small staff of the ablest consulting engineers whose services could be secured need not be "justified" to any engineer or man of affairs who has had a large measure of responsibility for any great construction enterprise, public or private. The safety and economy resulting from having such advisers constantly at hand will be evident to any unprejudiced person who will take the pains to be thoroughly informed.

The pity is that a group of intelligent men could have been party to an attack upon men of high character in public service, endeavoring to discharge their duties faithfully amid difficulties. For men thus maligned, unfortunately, there is small redress.

The Problem of Unemployment

IN former times, when most of the population lived upon farms or was engaged in small and local industries, every man was near his natural job and could generally find work. In the development of civilization there have come to be great enterprises which upset the simple life and often separate a man from his natural labor. Especially when these enterprises are intermittent in character, the problem of keeping the men busy between times, and of getting them shifted from one job to another, is very difficult.

In such enterprises as the recently completed Los Angeles aqueduct, the new Catskill water supply of New York City and the Panama Canal, thousands of men are brought together, organized, trained and made efficient. These include employees ranging from the chief engineer to the laborer and the water boy. The contractors, with their forces, are included, and also, in a way, those who furnish supplies for the work or to the employees. When the work is completed this army is disbanded. It must find its way gradually and clumsily into other organizations and other enterprises. The economic loss of an efficient organization alone is great, but the loss to the men themselves may be greater. The period of uncertainty, the lack of employment, and finally the getting of work not well suited to them are demoralizing.

If the break-up comes in good times, it is hard enough. If it comes in bad times, the case is indeed a sorry one for many a worthy and efficient employee. As a man's job is specialized and higher, it is harder for him to get placed again, and for that reason no class of employees suffer more than the engineers.

Here is food for careful thought. Have we reached the best organization that is possible, and if not, to whom may we look for help in keeping the people busy? Who will take up the splendid material and reorganize it, and work it in on some job that is useful, and that will have been worth doing when it is finished?

Among those thrown out of employment at the completion of any large undertaking there are always some who have capacity, and sometimes great capacity, for finding new jobs. If jobs do not exist, they stir up work not previously contemplated; they create the jobs. The more enterprising contractors and engineers lead in this. The country is canvassed for possibilities of doing things to meet needs not yet felt. The primary idea is to create work. If someone can be got to pay for it, the usefulness of the finished structure, desirable as it may be, is secondary.

In a new country, with comparatively undeveloped natural resources, the opportunities for doing useful work are almost unlimited. As long as the money for these enterprises is furnished by private capital, through corporations or otherwise, there is a natural check on the character of the work undertaken, for, in general, money will not be invested unless there is a prospect of profit. For a profit to be earned two conditions are necessary—the business must be a profitable one and the Governmental conditions under which the business will be permitted to exist must be favorable. In the past, when great municipal, State or Government enterprises have been completed, the men whose services were no longer needed have had chances of employment in various private enterprises. Railroads, electric lines, water-power, drainage and irrigation projects were under way. But if the Government goes on with larger and larger projects, and if private capital is restricted in its fields and in its methods, until it will no longer be attracted to new projects, the balance wheel furnished by corporate employment will be crippled when it is most needed.

These reflections are none too pleasant for him who believes that the welfare of the country depends upon keeping the people engaged at all times, and in tasks for which they are well fitted. Discussion of the subject is needed, and has not been wanting during the past winter. There seems, however, to be a neglect of the fundamental economic aspects. Thus far the alleviating measures have been confined to labor bureaus by which the man and the job are brought together. That is a phase that needs attention. But unless the general economic situation, the possibility of keeping private capital continuously and profitably employed, is taken into account, the fundamental cause of unemployment will escape those who are working for its elimination.

Proposed Code of Ethics for Civil Engineers

THE proposal by the board of direction of the American Society of Civil Engineers of a code of ethics is a most important step, the advisability of which has long been discussed. Since the question was first mooted by the civil engineers, the American Institute of Electrical Engineers has adopted a code, while the American Society of Mechanical Engineers has had a draft under discussion for over a year. Both of these documents are long, going into details as to the relations of the engineer to his client or employer, to the public, to other engineers, to the ownership of patents, and finally to certain standards set up for the profession. The two codes are quite similar, that of the mechanical engineers being based on the code of their electrical brethren. By contrast the proposed code for the civil engineers is a very simple document. It runs as follows:

"It shall be considered unprofessional and inconsistent with honorable and dignified bearing for any member of the American Society of Civil Engineers:

"1. To act for his clients in professional matters otherwise than as a faithful agent or trustee, or to accept any remuneration other than his stated charges for services rendered his clients.

"2. To attempt to injure falsely or maliciously, directly or indirectly, the professional reputation, prospects, or business, of another engineer.

"3. To attempt to supplant another engineer after definite steps have been taken toward his employment.

"4. To compete with another engineer for employment on the basis of professional charges, by reducing his usual charges and in this manner attempting to underbid after being informed of the charges named by another.

"5. To review the work of another engineer for the same client, except with the knowledge or consent of such engineer, or unless the connection of such engineer with the work has been terminated.

"6. To advertise in self-laudatory language, or in any other manner derogatory to the dignity of the profession."

Undoubtedly this draft, passed by the Board of Direction and ordered submitted to the Baltimore convention with the recommendation that it be adopted by the society, will be both commended and criticised for its length. In such small compass details had to be omitted. On the other hand, there is far more likelihood of this draft being adopted because of its brevity than if, like the codes of the mechanical and electrical engineers, it attempted not only to cover ethical questions, but also to lay down general rules of practice which have no ethical bearing. The difficulties of the other two national societies in securing the adoption of their codes probably has been due to this mistaken endeavor to cover too much.

It will be noted that all of the rules except the first and the sixth refer to the engineer's relations to other members of the profession. This is an admission that the unprofessional conduct is more likely to be found in the dealings of engineers with each other than in their outside relations. Reflection will convince one that this is probably correct, for outside of complaints as to low fees the chief criticism leveled at engineers is due to fishmonger methods of trying to get work.

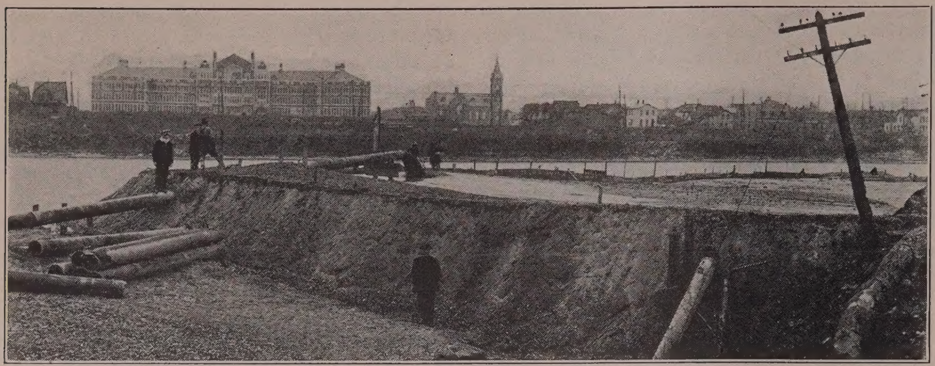
It is apparent that the code has been very largely drawn up with the consulting engineer in mind, and the criticism, therefore, must lie that the document affects only a limited number of members of the society. This necessary limitation of a code has long been appreciated. The membership of the society is so heterogeneous, composed of employed engineers, consulting engineers, contractors and manufacturers, that no single code can fit all. Nevertheless, this fact does not constitute a valid argument against the adoption of any code whatsoever. A code limited to a few members is better than none at all. Manifestly members are subject to it only in so far as it covers their relations.

The discussion on this draft, which should be full and thorough, will undoubtedly bring forth suggestions as to additional sections. Very likely, however, most of these will be found to relate to details or to be such as are indisputably covered by legal decisions or the common rules of right and wrong. For instance, any declaration as to the ownership of designs as between an employed engineer and his employer would be superfluous. Legal decisions have settled that. Moreover, the acceptance of a retainer on a work that in itself is vicious need not be covered in a code.

There is, however, in the judgment of this journal one additional provision that should be most seriously considered. The code provides no machinery for its enforcement, nor does the constitution of the society lay down a procedure which gives the code real teeth. If adopted as it is, without amendment of the constitution, it will be a fine declaration, but with only such moral effect as its occasional reading may have on the members. If, however, it were clearly understood that the code was to be enforced, that charges were to be brought under it, it would quickly have a salutary influence on lax members of the society.

It is true that ten members may now prefer charges of unprofessional conduct against any member, whereupon the board of direction will make an investigation. Experience has shown, however, that this provision of the constitution is seldom called into use, even though mutterings against members of the society are not infrequent. If, on the other hand, there were a grievance committee, whose sole duty it would be to investigate charges when preferred by two or more members, there would be more of a tendency to bring matters to proper attention. The American Institute of Architects, appreciating this, has a grievance committee, known as the committee on practice. The board of directors, the executive committee, any chapter or any member may bring to the attention of this committee "any alleged unprofessional conduct on the part of any member, without being deemed to have entered a formal complaint against such member." The committee makes a preliminary examination, and if the facts so warrant presents the case to the judiciary committee, which has power to decide the case. The board of direction, therefore, are relieved of this work, and act only, as a court of last resort, on appeals. The effect in the architects' institute has been excellent, and the position the institute occupies today is due to the fact that its codes are not merely idealistic documents, to be framed alongside the parlor mottoes, but expressions of principles that the organization is determined to enforce.

In governmental matters a law is worthless unless it carries a penalty for its violation, and unless there is machinery for enforcing it. In fact these are fundamental principles. Law without enforcement is ridiculous. So, too, in society matters. Principles and codes, unenforced, become laughing stocks, worse than nothing. Machinery for the enforcement of the civil engineer's code, therefore, if adopted, is certainly worthy of the most serious consideration.



Placing Hydraulic Fill and Sustaining It with Revetments

Building New Pennsylvania Bridge over the Maumee River at Toledo in Record Time

Part I—Preparing Subsurface with Suction Dredges and Bucket Excavators and Driving Piles in Deep Water with a Follower

By R. C. MILLER
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REPLACEMENT of the old drawbridge of the Pennsylvania Lines West of Pittsburgh over the Maumee River at Toledo, Ohio, with an independent parallel structure has just been completed in the record time of eight months and one day. The general contract was let last July and the bridge was put into service April 2, one day after the opening of navigation. To bring about this result it was necessary to use the wet method of substructure construction without pumping the open caissons, and employ numerous other time-saving expedients. The present article will confine itself to the excavation work and the placing of the pile foundations.

Toledo is located about 4 miles up the Maumee River from its mouth. Through the city the river is about 1200 ft. wide, and is 15 ft. deep outside of the navigable channel, which has a depth of 23 ft. The normal variation of water level is about 2 ft., although with certain winds the water may be blown out into Lake Erie, lowering the river a maximum of 6 ft., and in times of freshet, with ice floes, the water often reaches an elevation of 10 ft. above normal. On account of the closeness of the lake, which offers a clean outlet, the current is sluggish under normal conditions.

DIFFERENT TYPES OF BRIDGES

Between the city and the bay there are four railroad bridges and three city bridges, all of which have draw spans to take care of the lake traffic. The early bridges contained little masonry, the stone work extending about 3 ft. under mean lake level and resting on a timber grillage which was supported upon piling. In later bridges the piles were cut off at the top of the blue clay, near the river bottom, and covered with timber grillage, the masonry, therefore, extending to the river bed. Present practice, made possible by the general use of concrete, encases the tops of the foundation piles for a depth of 3 or 4 ft., no timber grillage being used. While the design of the new Pennsylvania bridge places it in this third group, the wet method of construction and the manner of driving the piles really place it in a fourth class, in that the same results were obtained in about five months which with ordinary methods would have taken at least a year.

General dimensions of the new bridge and its relation to the old are shown on the plan. The short girder span at the extreme east end was added, at the request of the Government, after the construction was under way, the floods of March, 1913, having raised the question of a sufficient length of structures over the larger rivers. For construction reasons the new bridge is built on a parallel offset line, 65 ft. downstream from the old structure. The alignment is tangent, ascending easterly on a 0.6-per cent grade.

The eight concrete supporting piers and two abutments extend 4 ft. below the top of the blue clay and rest upon oak piling approximately 25 ft. long. The concrete in the fender piers does not extend to the river bottom, but is supported about 12 ft. under mean lake level by oak piling surrounded by 2-in. limestone held in place by interlocked steel sheet piling, which gives a good foundation for the concrete and still allows enough elasticity in the pier to absorb the shock of a collision.

Because of the river traffic and the fact that the old and new draw spans would not clear each other, it was necessary to erect the steel work, especially the swing span, during the closed season or before April 1, 1914, and since the contract was not awarded for the substructure until late in July, 1913, the substructure had to be completed in about five months, or by Dec. 31, in order to allow sufficient time for the erection of the superstructure. Another reason for rushing the masonry was to escape the cold winter usually experienced in this locality after Jan. 1, which would naturally retard the progress of any uncompleted work.

GENERAL PROCEDURE

The length of time required to construct the eight piers and two abutments by the use of water-tight cofferdams, considering the great depth, argued in favor of open caissons of steel sheet piling, all work being done under the water. The general scheme of operations was as follows: As much of the foundation excavation as was possible, considering the tendency of the material to slip, was removed by a hydraulic dredge. Interlocked steel sheet piling was then driven in open-caisson form, and the remainder of the excavation removed by a clamshell or orange-peel bucket operated from a derrick scow. After sufficient depth was obtained, the foundation piles were driven through an open tube and then the concrete blanket or footing course placed, the piling projecting into the blanket about 3 ft. The steel sheet piling acted as a form to retain the footing course when placed.

After the footing course had set sufficiently, the forms for the neat work were placed inside the caisson, and in the case of the wooden forms, weighted down, and the neat work poured without resorting to pumping. The sheet piling prevented any current in the water, which was practically still at all times. After each pier was completed, the sheet piling was pulled by a pile driver equipped for this service. The entire operation was pushed with both day and night work, practically two complete organizations being required.

Approximate quantities in the work were

One Floating and Two Roller Pile Drivers Seen in Action as follows: Foundation excavation, 11,000 cu. yd.; piling, 39,000 lin. ft.; concrete, 13,650 cu. yd.; hydraulic fill at east end, 50,000 cu. yd.

FOUNDATION EXCAVATION

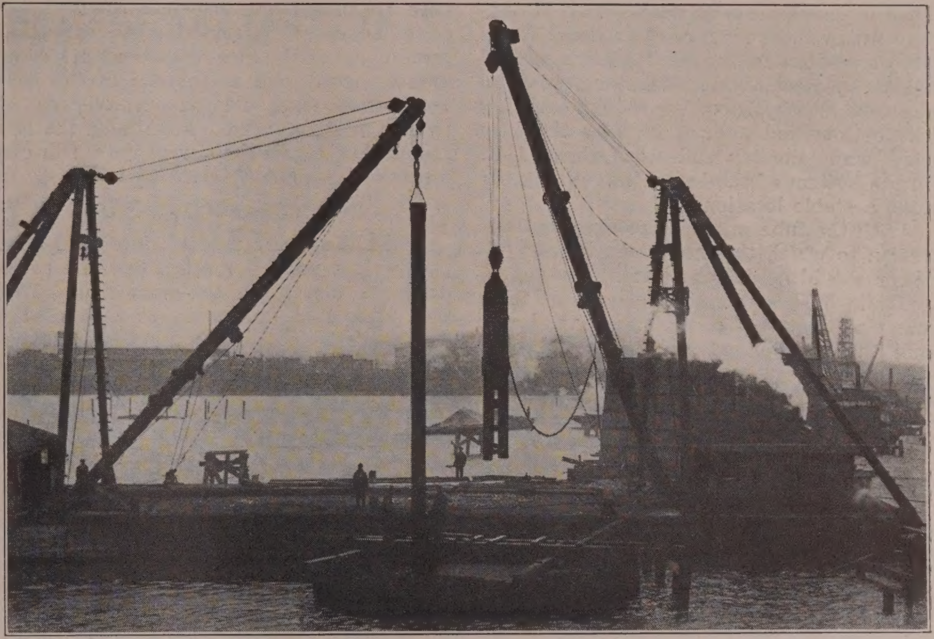
In connection with the change in line of the bridge, it was necessary to construct at the east end a fill about 1000 ft. long and from 18 to 25 ft. high. On account of its location and lack of other filling material, it was decided to make this fill with a hydraulic dredge. This plan worked in well with the foundation excavation. A 24-in. hydraulic dredge started at the east side of the traveled channel at the east end of the draw and dug a swath to the east abutment 100 ft. wide and from 25 to 30 ft. below mean lake level, depositing the material by means of pontoons in the fill at the east end of the bridge. This first cut practically cleaned off the blue clay so that further excavation at the site of the piers could be made with a clam shell outfit.

The ashlar masonry piers of the old bridge, built about 1876 with the timber grillage just below mean lake level, was supported by piling. Incomplete records indicated that the bottom of this piling was at a higher elevation than the proposed bottom of the foundation excavation for the new bridge. Considering the class of material on top of the blue clay—a fine, sandy, muddy silt—it was necessary to protect the old masonry before the foundation excavation was removed to proper depth, lest this deposit should slip into the hole and carry with it the support of the old masonry.

To protect this masonry ½-in. Lackawanna interlocked steel sheet piling 45 ft. long, the type used for the new piers, was driven in the shape of a half moon around the downstream ends of those piers which were within the distance of a 1½ to 1 slope of the proposed excavation, and held in place at the top by 1-in. steel cables passing around clumps of piles driven at the up-stream ends of the old piers, thus taking all strain from the old masonry and at the same time holding the material around it in place.

HYDRAULIC DREDGING

The bottom of the footing course was about 4 ft. below the top of the blue clay, being from 38 to 40 ft. below mean lake level. The maximum working depth of the hydraulic dredge was 32 ft., and it was intended to complete the remaining 6 to 10 ft. of excavation with a clam shell or orange-peel bucket, after which the foundation piles were to be driven with a floating driver, after which the sheet piling was to be driven.


Driving Piles for Pivot Pier with Scow with Two Derricks

On account of the under current in the river and the light sandy deposit which covered its bed, the foundation holes were filled with silt from 2 to 3 ft. deep over night. This showed this scheme to be impracticable, since it was feared that the under current, together with the agitation from driving, would deposit material around the tops of the foundation piles, which were to extend into the masonry about 3 ft. for proper bond, and that this material could not easily be removed. This condition, therefore, changed the method, and it was decided to drive the open caisson immediately after the hydraulic dredge had cleaned off the blue clay, taking out the remaining excavation with a clamshell outfit, and driving the foundation piling inside the caisson with land drivers resting on top of the falsework which it was necessary to construct along two sides outside of the caisson. This arrangement was carried out and was entirely satisfactory.

An idea of the class of blue-clay foundation encountered is best illustrated by the fact that for the last foot or so an ordinary orange-peel bucket would not pick up the material and it was necessary to use one of the heaviest types of clamshell, fitted with long teeth. Even this arrangement was so slow that it was necessary to use dynamite in some of the piers. In every foundation the last 2 ft. of excavation cost practically as much to remove as all excavation above it.

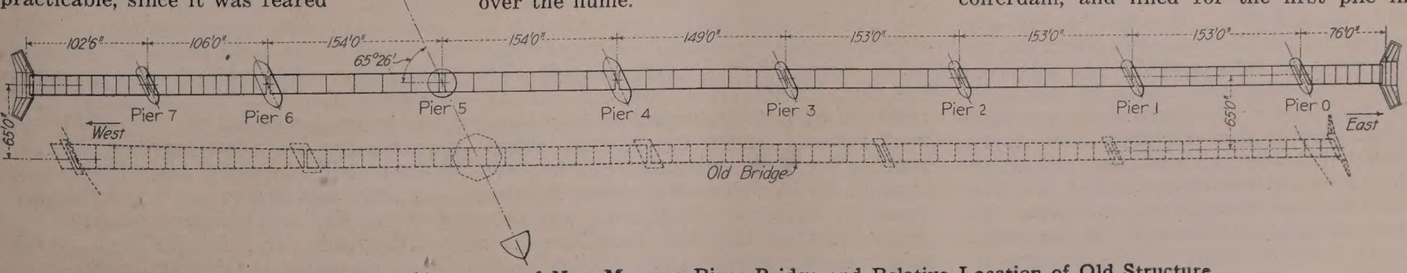
Only a small part of the total fill required could be made from the foundation excavation. The remainder was made with a 15-in. hydraulic dredge, sucking the material from the bottom of the river below the location of the bridge, carrying it a distance of from 100 to 500 ft. and lifting it to a maximum height of about 25 ft. above mean lake level, the fill being coaxed up and retained at approximately a 1½ to 1 slope by means of a series of revetments. Only a small amount of material was lost over the flume.

Peeled oak piles were used for the foundations, their length varying from 19 to 25 ft. according to the distance they could be driven into the blue clay. The tops were driven to an elevation of about 3 ft. above the bottom of the footing course in order to give a bond in the concrete. Land drivers with 70-ft. leads were used, the piles being driven through a 19-in. steel tube 45 ft. long, using a 46-ft. wood follower.

DRIVING THE PILES

As soon as the excavation was completed and the necessary falsework constructed along two sides of the cofferdam the entire layout of piles, which were spaced from 2½ to 3 ft. according to the loads, was made on the falsework in such a manner that a series of lines stretched across the open caisson on the center lines of the respective rows would by their intersections give the exact center of every pile to be driven. Thirty-foot 12 x 12-in. timbers were laid across the cofferdam between each row of piles with one face of the timber located so that when the driving tube was hung in the leads alongside it the center of the tube would coincide with the center of the row. These timbers remained stationary during the entire driving operation for each pier, the 90-deg. lines being stretched and intersection points marked on each timber, thus giving a very accurate and at the same time permanent location for each pile. Other 12 x 12-in. timbers were then placed on top of these timbers, at right angles to them, on about 8-ft. centers, care being used to place them between the located rows of piles.

Upon this as a foundation a roller driver was drawn, generally from the end of the cofferdam, and lined for the first pile in



Location of Piers and Abutments of New Maumee River Bridge and Relative Location of Old Structure such a manner that it would drive the entire row without moving the rollers. With the driver located, the steel tube was picked up at one end, swung into the leads and dropped into position, its weight carrying the bottom end 3 or 4 in. into the blue clay, while the top end stood about 4 ft. in the bottom of the leads, thus giving the tube a stable location.

With the tube in place the computed elevation to which the top of the timber follower should be driven to bring the top of the pile 3 ft. out of the clay foundation was plainly marked on the side of the leads, and the pile, peeled and sharpened, was picked up and dropped into the tube. This was followed by the timber follower, which was just large enough to fit neatly inside of the tube. The follower had a steel sleeve on the bottom which encased 18 in. of the top of the pile and held it true. Follower and pile were then dropped to the bottom, thus expelling the water through the top of the pipe. The hammer was lowered to place and the pile driven to resistance.

In determining the point of resistance, care was used not to broom the top of the pile, and it developed that a very good guide against this was obtained by paying close attention to the top of the follower exposed to view. Subsequent inspection of the foundation piling of each pier made by two separate divers, one for the contractor and one for the railroad, showed that of the total 39,000 lin. ft. driven, the tops of only five were shattered or otherwise injured.

With the pile driven home, the hammer was raised and the correct length of the next pile determined from the action of the pile just driven, so that when driven to resistance its top would be at the right elevation. Follower and tube were then picked up and the driver was lined up for the next pile. The driving level once established was generally good for the entire foundation.

PILING CHART

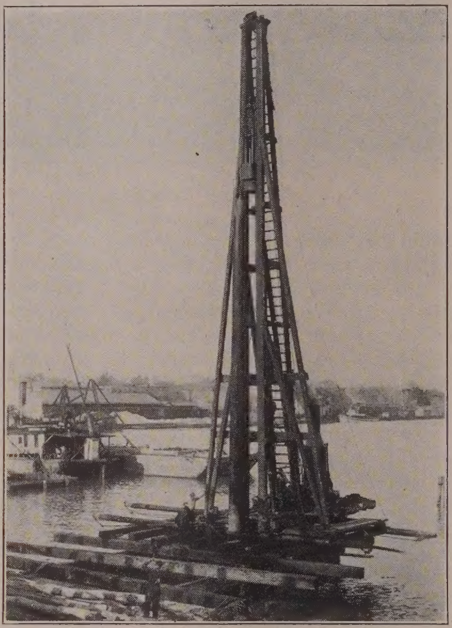
The inspector in charge was furnished a chart of convenient size showing the location of all piles in the foundation, and as each was driven, he checked it off, and at the same time recorded its length and the location of its top with respect to the driving level. In this way he was able by watching each pile as driven to vary the length of the piling according to the resistance encountered, and thus keep the actual tops within a variation of 6 in. from the driving level established, at the same time driving each pile to resistance. Since with the removal of the follower and tube from the driven pile all traces of the finished work were lost to the eye, this chart was invaluable and prevented any possible chance of error. It also furnished a full record of the foundation when completed.

In driving the piling in the first foundation it was felt that the compression of the clay due to the large number of piles driven would be sufficient to raise the elevation of the clay and thus frustrate the 3-ft. provision for bond between the piling and the concrete. To safeguard this and determine just what allowance should be made, the excavation for this foundation was made 2 ft. deeper than called for, but it was found that after all piles were driven the blue clay had risen only about 6 in. This allowance was made in the remainder of the foundations, and proved to be the correct figure for each pier.

Progress was very discouraging at first, only five piles being driven in the first day of ten hours. However, with the increasing familiarity of the men with the work came greater speed, and at the end of the first week they were driving an average of twenty piles in ten hours, while on the last foundation, eighty-four were driven in one continuous stretch of eighteen hours.

DRIVING DRAW-SPAN PILES

While the above method developed good progress, it was impractical to use a roller driver on pier 5, for the draw span. This pier is located in the direct path of the swing of the old draw, and the condition of the river traffic at this time required opening the draw at least twenty times each day. Removal of the driver would have been necessitated at each swing. Consequently the foundation piles in pier 5 were driven with swinging leads operated from a derrick scow lying at one side of the pier, in the clear of the open position of the draw, and blocking but one side of the channel. The outfit consisted of a derrick scow carrying two booms, one at each end. One boom handled the driving tube and placed the piling, while the other handled the suspended leads with steam hammer and timber follower. Considering the handicap of working in the path of a swinging draw and the difficulty of operating two booms, very good progress was made on this foundation, the 137 piles being driven in eighty hours.



45-Foot Tube Suspended in Leads

[In an early issue Mr. Miller will describe the form work and explain why wooden forms were found more satisfactory than the steel forms first tried.—EDITOR.]

INDIFFERENCE to sanitary conditions, accompanied by colossal ignorance, is the direct cause of the large amount of sickness now prevalent among many rural towns in Michigan, according to the views of Henry F. Vaughan, sanitary engineer of Detroit, expressed in a paper read recently before the Michigan Engineering Society. Unless there is improvement in the immediate future Mr. Vaughan fears an epidemic of typhoid or some other disease. He feels that there is a great need for money and its application by competent engineers.

Goethals for Full Tolls

Opposes Exemption, as Canal Will Need All Revenues It Can Get

GOV. GEORGE W. GOETHALS of the Canal Zone is opposed to tolls exemption. His views, as set forth in a telegram to Major F. C. Boggs, were given to the Senate Committee on Interoceanic Canals on April 24. The telegram reads:

"For Senator Simmons: My views on economic aspect of the tolls question follow:

"Government will expend $375,000,000 for canal and intends to charge for its use. Basis of tolls given in Emory Johnson's report on Panama Canal traffic and tolls, pages 193, 194 and 195. These figures should be corrected for operating expenses, which will be $500,000, including depreciation. I believe tolls should pay outstanding indebtedness, fixed charges and operating expenses, leaving amounts appropriated from current revenues to be contribution of present generation toward canal, and because of unknown future conditions not burdening posterity for remainder.

"Bond issues to date under canal acts, $134,651,980; interest on these approximately $3,200,000; annual amount to be set aside for redemption of bonds, $3,250,000; operation and maintenance, $5,250,000; annual payment to Panama, $250,000. Amount to be secured annually through tolls is, therefore, $11,950,000. If additional bonds are issued these figures should be changed accordingly.

"To secure this amount through tolls it is necessary to charge all shipping as indicated in report quoted. Established steamship companies fix rates after conference, and as they are in business for profit, will demand the highest rates the traffic will bear. These rates will be independent of tolls. It follows that exemptions from tolls will not give cheaper rates from coast to coast to either shipper or receiver, but will increase profits to shipping companies.

"The same is applicable to rates from interior points to either coast. Agreements will be made between railroads and ships for through rail and water rates, same as at present, rates divided between two interests as per agreement.

"Again, exempted tolls will not give lower rates to shipper or receiver. Therefore, free tolls to vessels engaged in coastwise trade result in a subsidy to a class of shipping already fully protected and not in need of subsidy.

"I do not believe in exemption of tolls for coastwise trade, first, because this amounts to a subsidy to a class of shipping and will benefit stockholders and not shippers; second, because this canal will need all revenue it can get to pay its current expenses and indebtedness. GOETHALS."

EXPRESS motor truck service has been inaugurated in Chicago with three 2-ton and one 5-ton motor trucks for the municipal bureau of engineering. Formerly if one of the eight district offices of the water pipe extension division required two pigs of lead and half a dozen shovels a team had to be dispatched to the central store house. The order has just been issued to each division head by John Ericson, city engineer, that orders are to be placed the previous day and a charge per hour of $2 will be made for a full load and prorated when several deliveries are made on the same trip.

Need for Simpler and More Definite Water Laws

Difficulty of Changing Present Practice Discussed, with Citations Showing Explicitness of Tennessee Decisions

By ROBERT E. HORTON
Consulting Hydraulic Engineer, Albany, N. Y.

THE fundamental principles of water law in most of the non-irrigated States seem to be fairly simple and well established, so far as the right to the use of water by the riparian owner is concerned. In this respect the definite principles may be simmered down briefly to the doctrine that the owner of the bed and banks of a private stream is entitled to have the stream flow through or past his property in its natural order and régimen and with its quality not artificially deteriorated.

As regards boundaries of land adjacent to streams, the fundamental doctrines are by no means so simple or universally accepted. In some cases lands are deeded to the thread or center line of a stream; in other cases they are deeded to high-water or low-water mark or to the bank of the stream at its ordinary stage. Inasmuch as the high and low-water marks and ordinary stage of a stream are seldom the same in two successive years, and even the thread of the stream may vary in position or its location be subject to dispute, definitions of land boundaries involving these terms have given rise to a great deal of litigation and to an almost endless variety of decisions, some seemingly contrary to others.

Again, the question has arisen many times as to what constitutes a navigable stream. There is a diversity of the findings of State and Federal courts in this regard, and even a wide diversity in opinions of the courts as to different streams within the same State.

DIFFICULTIES IN REVISING LAWS

In view of the uncertainties as to the relations existing between different riparian owners and between riparian owners and the State at large, and the frequent and costly litigation which has resulted from these differences of opinion, efforts have been made in the State of New York to revise the whole scheme of water law and get it on a simpler and more definite basis. Thus far this has been found impracticable, and probably cannot be accomplished through legislation alone or by any method in a short space of time. Such changes as seem necessary would require for their permanent establishment a line of consistent rulings by the Court of Appeals. Many precedents would necessarily be abolished and existing rights on which enormous investments have been incurred would perhaps be jeopardized. Apparently in regions like New York and New England, where the development of riparian rights has already been carried on to a large extent and titles and decisions are long standing, substantial improvement in water law in the broadest sense can only be accomplished by a gradual education of the public, the legal and engineering professions and the courts to a better understanding of the conditions. Even then to accomplish definite results it will be necessary that the courts of last resort follow well-defined principles of equity and justice, laid down in accordance with hydraulic science, and in accordance with the marked changes in conditions which have resulted from the advent of hydroelectric-power development. This must be done with a certain amount of disregard for established precedent, and inasmuch as the basis of the decisions of the high courts has heretofore usually been that of established precedent, it is evident that the changes can only be brought about gradually and with difficulty.

In newer States where development of riparian rights is only just being undertaken or is comparatively new it would appear that a different set of conditions exists and that if the matter is taken in hand in time a great deal of good can be accomplished by preventing the growth of such a complicated and unsatisfactory system as exists in the older regions of the country.

SIMPLE AND EXPLICIT DECISIONS IN TENNESSEE

Something has already been accomplished along this line. In the State of Tennessee, for example, there is a simple and seemingly easily workable law of condemnation of lands, intended to apply especially to takings of flowage rights, rights-of-way for transmission lines, etc. This law does not require the costly condemnation proceedings through the Supreme Court which are required in New York State, but permits of the taking of small parcels of land on an appraisal by a board of three land viewers without the taking of testimony, in very much the same way that line-fence disputes are settled in the Middle West by so-called fence viewers, without further court proceedings. The owner of the property is protected against injustice by the right to have the decision of the land viewers passed on by the higher courts.

In regard to what constitutes the thread of a stream and what constitutes a navigable stream, the decisions in the State of Tennessee seem to be remarkably simple and explicit, as illustrated by the following decisions by the court of last resort of the State:

"*Not Navigable.*—A stream which is not of sufficient depth naturally for valuable floatage, such as rafts, flatboats and small vessels of light draft, is not navigable in any sense, strict, legal or ordinary."—Irwin v. Brown, 3, Shan. Cas. 209; Allison v. Davidson (Cha. App.) 39, S. W. Rep. 905.

"*Riparian Owners, Rights of, Boundary.*—The grant of land by the State, bounded by, on or along a stream not navigable, carries title to the center, or thread of the stream, as to an island lying next to the bank between it and the middle line."—Branham v. Turnpike Co., 1 Lea, 706; citing 2 Swan, 10.

"The same rule applied to private grants, where the grantor owns the entire stream."—Ibid.

"The thread of the stream is the middle line between the shores, irrespective of the depth of the channel, taking it in the natural and ordinary stages of the water at its medium height."—Ibid; Posey v. James, 7 Lea, 104.

"Owners of land on navigable streams have title to ordinary low water mark, when their papers call for the bank, unless a line is run and marked upon the bank."—Nance v. Womack, 2 Sha. Cas. 202.

"The case of Elder v. Burrus (6 Hum. 364), following the North Carolina case of Wilson v. Forbes (2 Dev. 364), settles, as the law of this State, that the owners of lands upon navigable streams have title to ordinary low-water mark, but if not navigable, then to the center of the stream."—Posey v. James, 7 Lea, 103.

"The general rule undoubtedly is that a call for the stream or bank, or object on the bank, and then with the stream, according to its meanders, will carry the boundary ad filum aquæ, whether the grant be by the State or a private person or individual."—Holbert v. Eadens, 5 Lea, 210; Massengill v. Boyles, 4 Hum. 205; Martin v. Nance, 3 Head, 650; Elder v. Burrus, 6 Hum. 358; Stuart v. Clark, 2 Swan 10; Branham v. Turnpike, 1 Lea, 706; Posey v. James, 7 Lea, 104; Spaine v. Railroad, 1 Shan. Cas. 183.

"This rule applies even when the title papers call for a corner on the bank above low-water mark, unless it is shown that a line was run and marked on the bank, as the true boundary."—3 Head, 650; 5 Lea, 210; 7 Lea, 102.

"Where alluvial accretions are made from the bank extending into the river, the title vests in the riparian owner."—7 Lea, 99.

"The soil below low-water mark on navigable streams in this State belongs to the public, and the title is invested in the State for the use of the public."—Goodwin v. Thompson, 15 Lea, 206.

"Title to soils under navigable streams cannot be acquired by individuals under our general land laws."—Ibid.

"The owner of land through which a stream passes may turn the waters from their channel for his use, provided he returns them to the channel before it leaves his lands."—Webster v. Fleming, 2 Hum, 519.

"A watercourse is defined to consist of bed, banks and water. It must be made to appear that the water usually flows through a regular channel with bank or sides."—State v. Pulp Co., 119 Tenn., 47, 104 S. W. 437.

"Where a deed of conveyance of land expressly calls for the water's edge at low water mark for the boundary, the rule that ordinarily applies to the rights of riparian owners in non-navigable waters does not apply, and the title does not extend to the center of the stream."—Webster v. Harris, 111 Tenn. 668, 69 S. W. 782.

QUESTIONS TO BE SETTLED BY HYDRAULIC ENGINEER

The determination of what constitutes the ordinary stage or the ordinary low-water mark or high-water mark of a stream seems to be fundamentally a question for hydraulic engineers and it would further seem that on streams where gaging records have been taken the answers to these questions can be worked out scientifically in a very definite manner, and that there should not be the difference of opinion between experts which commonly exists in technical matters, and which has led to the derogation of expert testimony in some quarters. It would seem that the ordinary stage of a stream is a matter capable of very explicit definition. Obviously it can be nothing more or less than the most usual stage of the stream. If a record of the stage or discharge of a stream for a period of years is arranged with the quantities for the different days placed in their order of magnitude, then it is evident that the middle or median quantity in the table will represent the ordinary stage or discharge, as the case may be. That this is the most usual stage or discharge is evident, since the stream is just as likely to be higher as lower, and vice versa. As to ordinary high water and ordinary low water, the finding of satisfactory definitions may appear a little more difficult. The writer has, however, used the following German definitions for these terms in a number of litigations:

"The ordinary stage is the median stage."

"Ordinary high water is the median point for stages or discharges of the stream which are above the median stage or discharge for the whole record."

"Ordinary low water is the median point for stages or discharges of the stream which are below the median stage for the whole record."

As a rule, the ordinary stage of a stream is less than the average or mean stage, and as a rule a stream is below its mean stage more than one-half of the time and above its mean stage less than one-half of the time.

Pneumatic Rammers for Concrete

It Is Claimed That They Produce Better Grade of Concrete at Less Cost Than Hand-Ramming

By CHARLES A. HIRSCHBERG
Of the Ingersoll-Rand Company, New York

PNEUMATIC rammers are used in all up-to-date foundries for tamping molds and have reduced labor and costs and improved the product. It is only recently, however, that they have been employed in such work as tamping earth around gas and water mains, building roads, constructing concrete buildings, making concrete pipe, artificial ashlars, steps and pillars and ramming concrete walls in tunnels.

Lower cost and greater speed are not the most important factors to be considered in this class of work. The improved quality of the pneumatically finished product over that rammed by hand is such that contractors, particularly in France, Germany and Belgium, are resorting to the use of the pneumatic rammer in the interests of safety and lasting attributes.

In the accompanying photograph are shown a number of Crown pneumatic rammers at work on a large concrete palace constructed last year at Nancy, France. This rammer is of the same general construction as the rammer used in foundry work, although the butt is different. In the drawing are shown several shapes of butts used for ramming concrete. The large butt is employed for general work and the small square one for working in corners and restricted areas. The round butt is used occasionally. Butts for ramming concrete are generally made of cast steel. For finishing the work and to avoid the spraying of the semi-liquid cement the butts are made of rubber, hard wood and occasionally of fiber.

The rammer is equipped with a trigger throttle by means of which its entire operation is controlled. In general construction this tool is simple; a valve of the spool type works in a hardened-steel valve box which is clamped between a cylinder and head block from which an extension tube or pipe leads to the throttle. There is also a piston, to the outer extremity of which a butt is attached. An air hose is connected at the top of the rammer. With the exception of the butt, all parts are generally made of steel. A tool such as that shown consumes approximately 24 cu. ft. of free air per minute at 80 lb. (with the throttle wide open) under which pressure it is usually operated. It strikes approximately 600 blows per minute at this pressure.

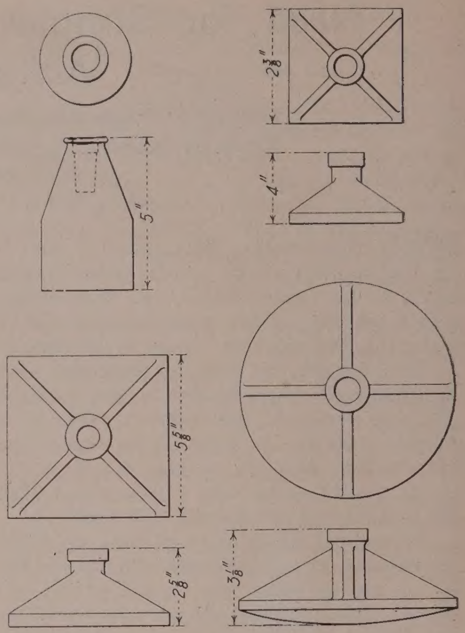
RECORDS OF PERFORMANCE

The average performance of one rammer operated by one man over a ten-hour period is 270 sq. ft. of surface rammed. In the making of big pillars it rams easily 212 cu. ft. in ten hours. The finished product is much superior in quality to that rammed by hand.

A Belgian engineer, who has made a study of this subject, found that in the ramming of concrete the number of blows struck played a far more important part than the power or strength of the blow. Such being the case the mechanical rammer, which strikes approximately six hundred light blows per minute, is ideal in comparison with hand work. In one experiment on comparatively small cubes it was demonstrated that sixty blows gave the concrete a resistance of 2844½ lb. per square inch; forty blows, 2460¾ lb. per square inch, and twenty blows, 1538 lb. per square inch. A similar test with hand-ramming gave a resistance of only 939 lb. per square inch. The hand rammer used weighed 33 lb. and was permitted to fall from a height of 20 in.

Expert workmen handling the hand rammer are unable to strike more than fifty blows per minute and maintain that speed for any length of time. It was shown that the same workmen using pneumatic rammers under an operating pressure of somewhat less than 80 lb. were able to maintain a maximum of four hundred blows per minute over a considerable period. It is also possible to vary the number of blows at will, reducing them to as few as three per minute.

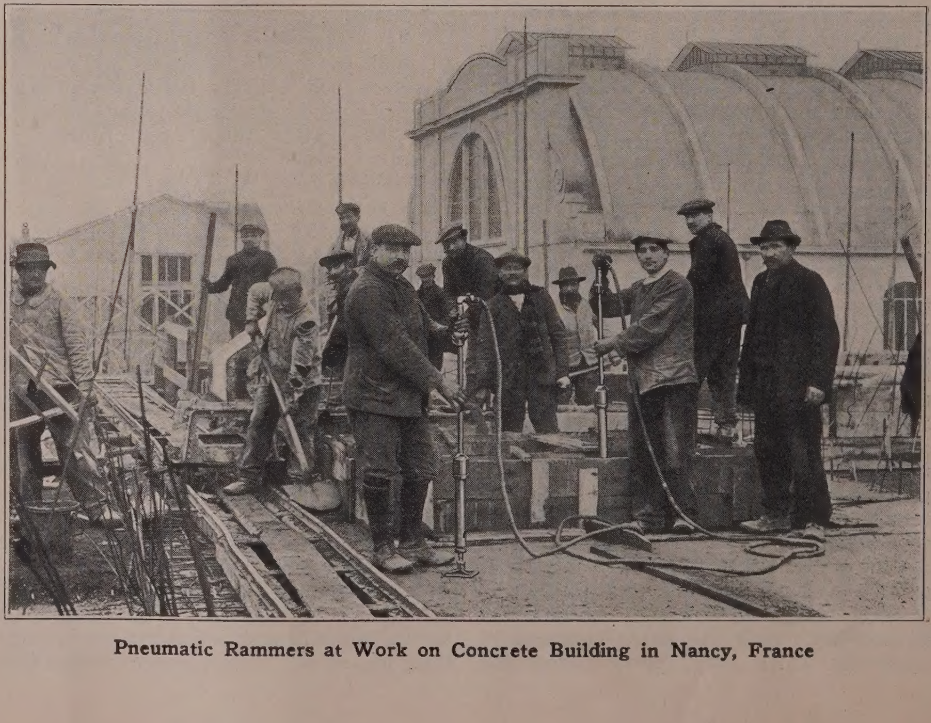
Further tests demonstrated that the pneumatically rammed concrete is much more dense than hand-rammed concrete and that one man and a pneumatic rammer would do approximately the work of five men working by hand. In the case of one study it was found that one man operating over a large area of concrete was able to cover 1076 sq. ft. of surface in ten hours. Experiments conducted in the manufacture of concrete pipe in the neighborhood of Paris showed a saving of 50 per cent in the cost of production.



Types of Rammer Butts

As on the majority of such work, air power is a part of the contractor's equipment, for the operation of temporary pneumatic lifts, riveters, etc., the use of pneumatic rammers does not entail any outlay for temporary power plant equipment and it would seem that the results obtained by their adoption are such as to warrant greater interest on the part of the contractor. The type of rammer illustrated in this article is manufactured by the Ingersoll-Rand Company.

WHARFAGE IMPROVEMENTS at Sydney are contemplated in a comprehensive scheme which, states a recent consular report, calls for the expenditure of $30,000,000 in a period of ten years.



Pneumatic Rammers at Work on Concrete Building in Nancy, France

Army Board Submits Flood Report

Outlines Policy to Prevent Recurrence of 1913 Disasters in Ohio Valley without Recommending Any Single Method of Prevention

NO single method of flood control to prevent a recurrence of the 1913 disasters in the Ohio Valley is recommended in the report of the special board of officers of the Corps of Engineers, U. S. Army, which was transmitted to the House of Representatives by Secretary of War Garrison on April 15. The board, appointed April 12, 1913, and consisting of Lieut.-Cols. Francis R. Shunk and Henry Jervey, Majors Charles S. Bromwell, John C. Oakes, Frederick W. Alstaetter and Lewis H. Rand, points out that reservoirs—both storage and impeding—unobstructed channels, and levees are all proper methods of river regulation, depending upon local conditions. One of the chief causes of floods is found to be obstructions in river channels. The board does not look with favor upon forestation and special methods of plowing as flood preventive measures. The report, published as House Document 914, Sixty-third Congress and abstracted below is an amplification of its first report, submitted May 26, 1913:

CAUSES OF FLOODS

The causes of floods in general are excessive precipitation, quick run-off, and inadequate carrying capacity of waterways. Methods of prevention of floods must, therefore, contemplate control of precipitation, control of run-off, or provision of adequate channels of escape. Damage by floods might also be largely prevented by another method not involving control of the flood itself, namely, by moving valuable property beyond the flood limits.

The method of moving property out of the way is that which will afford the most prompt and certain relief, and its adoption should be urged in all cases where at all practicable. The investigations of the board during the months of April and May, 1913, which consisted in personal inspections of sixty-six localities, showed that in every case a large percentage of the damage suffered was due to the fact that people had knowingly erected buildings and lived in territory which has always been subject to inundation by high water. It was evident that present conditions would have been far better if, in the past, when towns, roads and railroads, etc., were being located some effective authority had been exercised to regulate the placing of permanent structures on the lowlands of the rivers in the Ohio and Lake Erie basins. However, millions of dollars have been spent in improving lowlands subject to floods and in making encroachments which have so diminished the flood-carrying capacity of streams as to increase the height of floods and the velocity of flood currents. Some of this work can be undone, and in every case, where possible, it should be undone. On the other hand, the cost of completely remedying these mistakes—in some cases involving the moving of large portions of cities of 10,000 or more inhabitants, such as Cairo (14,548), Portsmouth (23,481), Marietta (12,923), Zanesville (28,026), etc., would be so enormous that it is evident that some form of flood control or flood protection will be much less expensive.

Forestation Controlling Precipitation—The only method of controlling precipitation which has been seriously proposed is that of forestation. The question has been debated at length in recent years and was considered by the National Waterways Commission in its recent investigations. Most authorities take the view that forests have small effect on rainfall; but the board would seem to be warranted in assuming that for all practical purposes the control of precipitation by means of forests is impossible and that the meteorological causes of floods are beyond human control.

CONTROLLING RUN-OFF

The methods proposed for controlling run-off are forestation, special methods of plowing, and reservoirs.

Forestation—This subject is discussed in the report of the National Waterways Commission on the effect of forests upon run-off and upon floods.

It is impossible to expect that any large amount of cultivated land can be returned to the state of forest, or even that further clearing tracts suitable for cultivation can be prevented. The board therefore concludes that reforestation is not a practicable method of controlling floods, and reserves for its final report the question of maintaining existing forests.

Plowing—Under this head may be classed contour plowing and deep plowing, either separately or in conjunction. It is not known that these ideas originated with the persons who have called them to the attention of the board; apparently the fundamental principles have long been known and practised in sundry parts of the world.

From its investigations and the collation of existing information the board feels justified in eliminating forestation and special plowing as methods to be undertaken by the United States. There remain to be considered reservoirs, and the various ways of increasing the capacity of channels, as by levees, widening, etc. It is by one or other of these methods, or a combination of them, that the problem is to be solved if at all.

RESERVOIRS

Reservoirs act to prevent or diminish floods by holding out of the flood discharge a large percentage of the run-off. Considered as means of flood control they may be divided into two classes—impeding reservoirs and storage reservoirs.

Impeding Reservoirs—The reservoirs called by this name are automatic, having discharge channels, always open, so proportioned that the greatest flow through them, under the greatest head possible, will not exceed the carrying capacity of the stream below. They have been used to a limited extent in Germany, and have been considered in conjunction with other methods as a means of flood control at Columbus, Ohio, where, as the board believes, the combined system would have been successful. They have the advantage that their usefulness as flood preventers can not be diminished by other more or less incompatible uses. But they have the disadvantage that they do not give full control of the discharge, especially with respect to phenomena of interference with other streams. From an economic standpoint, as compared with storage reservoirs, they allow a great deal of valuable water to run to waste.

Storage Reservoirs—Storage reservoirs, the discharge from which is more or less completely under control, may be used for purposes other than the control of floods. Many have been built in the United States, but chiefly for power purposes, and as these have not been used for flood control they furnish no conclusive evidence of effect upon floods. Reservoirs have also been used, both in Europe and the United States, as aids to navigation, and, although the use for navigation is to a large extent incompatible with the use for flood control, they have doubtless a considerable and beneficial effect upon floods. Notably the great system in northern Minnesota, with a capacity of 94,000,000,000 cu. ft., although manipulated primarily for the improvement of navigation, may be shown to have had an important incidental influence in mitigating floods over a considerable area. The board has examined all available literature dealing with reservoirs, and is of opinion that the subject has been exhausted so far as existing information permits.

The board states as a result of its investigations that flood control by means of reservoirs can not be dismissed forthwith as impracticable. On the contrary, for particular localities, and especially for the smaller streams, reservoirs appear to promise good results. It is to be remarked that the control of a stream like the Ohio is a problem of far greater magnitude than those presented by the comparatively small streams in Europe, for instance, where reservoirs have been used with a measure of success.

AUXILIARY CHANNELS

Auxiliary channels are of two kinds, those which parallel the stream for a large part of its length, and those which cut across bends.

Parallel Channels — Parallel channels have been suggested for the Mississippi River, but not so far as known for any of the streams in the district under consideration of the board. The topography of the river valleys in the Ohio and Erie Basins does not lend itself readily to this method, but it may be found applicable in some cases.

Cut-offs—The board's investigations indicate that cut-offs usually give only local relief. They necessarily cause steeper slopes, higher current velocities, and consequent greater erosive power; and by delivering the water more rapidly at their lower ends have a tendency to aggravate floods at such points. However, in special cases they have great value, as indicated by the studies made at Columbus, Ohio.

The great advantage of levees is that they give quick relief, they are equally suitable for local and extended uses, and are the most immediately available form of protection against floods. Their advantages and disadvantages are well known and need not here be theoretically discussed, as they have had practical trial on a large scale both in the United States and other countries.

The board finds that where levees have been properly built in accordance with correct designs they have been almost uniformly successful. Their cost can be adjusted to the resources of the locality, by consideration of the area and value of the territory to be inclosed, the width of the low land that may be abandoned to the river, and the height of embankment required to give proper economic protection. It may be stated as certain that any successful method of flood control will include the use of levees on a considerable scale.

The necessity for providing and maintaining adequate channels for the flow of water is clearly shown by the investigations of the board. At almost all the places visited the damage done was largely caused by encroachments on the natural channels, whereby their capacity was made insufficient.

In justice to those who make encroachments on the flood channel it must be stated that in no case that has come before the board has it been shown that the fault was committed intentionally, and in only one instance that has been called to the attention of the board was antecedent objection made. The drainage basins delivering water to the points in question had areas of from several hundred to several thousand square miles; no adequate studies of the flood possibilities had ever been made, and all development along the river had been based upon insufficient data.

CONCLUSIONS AND RECOMMENDATIONS

The board has reached the decided conclusion that no one method of flood control can be recommended to the exclusion of others. Local conditions vary so greatly between several basins that what is suitable to one stream is likely to be unsuitable for another. In some places (as at Dayton) reservoirs appear to promise good results. The board is informed that this method has been recommended by the engineers employed by the city. In others (as Columbus) a system of levees and increased dimensions with a cut-off are preferred. In other cases, after preliminary examinations, other combinations seem desirable. Therefore the board reports that it is not possible to recommend what method shall be used in each particular stream without knowledge of further facts.

These facts are of two main classes: First, those bearing upon the quantity of water to be expected; and, second, those bearing upon the capacity of watercourses to carry it away. The most important of these facts are, of the first class, the greatest run-off from the drainage area, and of the second, the least discharge capacity of the drainage stream.

The board has considered many methods of flood protection and prevention, and concludes that:

1. Forestation and special methods of plowing are not usually productive of practical results to an extent that would make it profitable to consider their employment.

2. Reservoirs, both storage and impeding, may be used to advantage in certain localities, especially on the smaller streams and where good reservoir sites are obtainable at reasonable cost.

3. The method of moving structures and bulky material out of the flood area will afford prompt and certain relief both by saving the property moved and by reducing flood heights and velocities by removing the obstructions. This method should be adopted so far as practicable, and new obstructions should be prevented.

4. Auxiliary channels, both parallel channels and cut-offs, may be used to advantage in certain cases.

5. Levees when properly built in accordance with correct designs have been almost uniformly successful; they give quick relief and are equally suitable for local and extended uses.

6. No one method of flood control can be recommended to the exclusion of others. Local conditions vary so greatly between several basins that what is suitable to one stream is likely to be unsuitable for another.

PROTECTION OF MISSISSIPPI VALLEY

In the opinion of the Mississippi River Commission and of the Chief of Engineers United States Army, the only practicable method of flood protection in the lower Mississippi Valley is that by construction of an adequate system of well-designed levees supplemented by bank protection. The project for flood protection in the Sacramento Valley, Cal., contemplates an extensive use of auxiliary channels. The city of Galveston, Tex., secured flood protection by raising the grade of the city. For the Miami River, Ohio, it is understood that the engineers employed by the city of Dayton, Ohio, have recommended a system of reservoirs.

If this work be placed under the War Department on the same basis as the river and harbor work, with which it is intimately connected, it can be accomplished with no large initial expense. No new and expensive organization need be created. Each case requiring appropriation will be presented in an intelligent, concise and concrete manner for consideration by Congress, and with Congress will rest the decision as to which rivers shall be considered, and the rate at which they shall be appropriated for.

New York Building Heights Regulated by Law

Governor Glynn Signs Bill Authorizing Division of City into Industrial Zones

HEIGHTS of buildings in New York City are now subject to regulation by the Board of Estimate and Apportionment. As announced in the Current News section of the Enginering Record last week Governor Glynn has signed the Conkling Bill, amending the New York charter. The text of the new law follows:

The Board of Estimate and Apportionment shall have power to regulate and limit the height and bulk of buildings hereafter erected and to regulate and determine the area of yards, courts and other open spaces. The board may divide the city into districts of such number, shape and area as it may deem best suited to carry out the purposes of this section. The regulations as to the height and bulk of buildings and the area of yards, courts and other open spaces shall be uniform for each class of buildings throughout each district. The regulations in one or more districts may differ from those in other districts. Such regulations shall be designed to secure safety from fire and other dangers and to promote the public health and welfare, including, so far as conditions may permit, provision for adequate light, air and convenience of access. The board shall pay reasonable regard to the character of buildings erected in each district, the value of the land and the use to which it may be put to the end that such regulations may promote public health, safety and welfare and the most desirable use for which the land of each district may be adapted and may tend to conserve the value of buildings and enhance the value of land throughout the city.

The board shall appoint a commission to recommend the boundaries of districts and appropriate regulations to be enforced therein. Such commission shall make a tentative report and hold public hearings thereon at such times and places as said board shall require before submitting its final report. Said board shall not determine the boundaries of any district nor impose any regulation until after the final report of a commission so appointed. After such final report said board shall afford persons interested an opportunity to be heard at a time and place to be specified in a notice of hearing to be published for ten consecutive days in the "City Record."

LOCATION OF INDUSTRIES AND BUILDINGS

The Board of Estimate and Apportionment may regulate and restrict the location of trades and industries and the location of buildings designed for specified uses, and may divide the city into districts of such number, shape and area as it may deem best suited to carry out the purposes of this section. For each such district regulations may be imposed designating the trades and industries that shall be excluded or subjected to special regulations and designating the uses for which buildings may not be erected or altered. Such regulations shall be designed to promote the public health, safety and general welfare. The board shall give reasonable consideration, among other things, to the character of the district, its peculiar suitability for particular uses, the conservation of property values, and the direction of building development in accord with a well considered plan.

The board shall appoint a commission to recommend the boundaries of districts and appropriate regulations and restrictions to be imposed therein. Such commission shall make a tentative report and hold public hearings thereon before submitting its final report at such time as said board shall require. Said board shall not determine the boundaries of any district nor impose any regulations or restrictions until after the final report of a commission so appointed. After such final report said board shall afford persons interested an opportunity to be heard at a time and place to be specified in a notice of hearing to be published for ten consecutive days in the "City Record."

Bulkheads for Tunnel Forms

BULKHEADS for tunnel forms are always difficult to build because of the roughness of the rock profile. The Pneumatic Concrete Placing Company, of New York City, which is specializing in tunnel-lining work, has been experimenting to find a suitable way of building such bulkheads quickly and cheaply and has found that cement sacks three-quarters full of sand can be used successfully. The sacks are laid on top of each other in the same way as bricks in a wall and can be fitted into all crevasses of the rock, following closely the irregularities of the profile. It is stated that one unskilled laborer placing sacks replaces two carpenters building forms. A bulkhead formed in this way is said to be tight and is left in place the same length of time as a wooden one. When the sacks are removed the concrete surface is rough and gives a good bond with the adjacent section.

Regulation of Technical Competitions

Steps Taken by American Institute of Architects, with Comments as to the Advisability of Similar Regulations for Engineers

THE American Institute of Consulting Engineers met on April 13 in order to discuss the regulation of engineering competitions. Since the American Institute of Architects has long been wrestling with the subject of competitions and has evidently solved the difficulties, three prominent New York architects, Arnold W. Brunner, Robert D. Kohn and Charles Butler were invited to lay before the members the work of their organization. Most of the space is here given to their addresses, because the comments of the engineers who spoke were almost entirely confined to endorsements of the work of the architects and specific experiences of competition from manufacturers.

As a result of the discussion the Institute's committee on professional practice and ethics was directed to take under advisement the drafting of a code governing competitions.

Preceding the discussion, which was presided over by Frank W. Sprague, president of the institute, the secretary, Eugene W. Stern, read a communication from Henry W. Hodge, which is pertinent to the subject. Mr. Hodge's letter was as follows:

"It has become the practice of certain municipalities to advertise for engineers to submit designs for engineering projects, without agreeing to pay anything for the successful design, or to employ the designer as the engineer for the work. This practice has evidently become prevalent in England, as the Institution of Civil Engineers has sent the following circular to each of its members, under date of Feb. 18, 1914:

" 'The council have had their attention drawn for some time past to a practice which exists among Local Authorities of inviting by advertisement engineers to submit in competition with others their terms for preparing plans and proposals for certain engineering schemes, accompanied then or perhaps at a later stage by estimates of the cost of the works proposed.

" 'In the opinion of the council such a proceeding is very undesirable in the best interests of the Public Authorities themselves and is derogatory to the engineering profession. The council desire to express emphatically the repugnance with which they regard the practice in question. They have every confidence that the members of the institution will support them by declining to respond in any way to such advertisements as those alluded to.

" 'The council have informed the Local Government Board of their action in making this communication to the institution.'

"A particularly unsatisfactory example of this procedure was shown by the city of Chattanooga, which advertised last January for designs for a bridge across the Tennessee River. They furnished no data whatever, and made no agreement as to accepting the best design, or in any way remunerating any of the participants. Unfortunately several reputable engineers spent their time and money in securing the necessary data and submitting designs, and after these engineers and several contracting companies had spent a week in Chattanooga advocating their designs the city decided to indefinitely postpone action.

"While this is the treatment they might have expected under such an advertisement, yet it appears that engineers of standing do the profession, as well as themselves, an injury in furnishing valuable information in competition, unless such competitions are surrounded by the proper conditions, giving the competitors the data on which to base their designs, arranging for the proper appointment of competent judges, and agreeing to retain the engineer making the selected design.

"I, therefore, recommend that the meeting pass a resolution requesting the council to prepare a set of regulations to cover engineering competitions, somewhat on the lines of those adopted by the American Institute of Architects, and to publish same in the engineering papers, and send same to every member of the American Institute of Consulting Engineers, requesting them to enter no competitions, except under such regulations."

INTRODUCTORY REMARKS

Mr. Sprague then introduced the subject of the evening.

I think we all recognize, he said, that the tendency in this country at present is to get as much engineering service as possible gratis. Perhaps that is in line with another class of would-be clients, the railroads, who are inclined to lay the burden upon the manufacturing companies, which seem to be pretty well equipped to make reports. I think this tendency is growing all the time. That it should grow in the case of municipal bodies is natural; they are generally subservient to politicians. With large manufacturing corporations, bridge builders and electrical manufacturing companies, that tendency also is growing. Electrical corporations are dealing with extraordinarily complex subjects. They have large corps of engineers who are designing their apparatus and they are well posted in regard to their particular product and equipment. They have been forced into the engineering field by competition and it is almost inevitable that they are now doing a great deal of this work.

We have with us to-night some guests representing, among others, the American Institute of Architects, who have settled these questions in their own way very satisfactorily. I am going to ask Mr. Brunner if he will give us the benefit of their experience in dealing with these problems.

REMARKS BY MR. BRUNNER

Mr. Brunner remarked that the engineers are confronted with the very same problems that have troubled the architects. They, however, have succeeded fairly well in solving them.

We have insisted, he said, upon being professional men—not salesmen selling goods. It is perfectly obvious, I think, that there is a difference between us and a salesman. In the building concerns, some of them will tell you that you do not need an architect or engineer. They will design the building, finance it, build it and turn it over to you with the key in the door. There are architects to-day who are advertising that they will do this, and I believe they are doing pretty well. The public, however, has begun to learn that if they go to a doctor, they had just as soon he was not interested in the corner drug store.

We are continually receiving invitations to submit plans and specifications containing the clause reserving the right to reject any or all bids, and these go promptly into our waste basket. We do not take any notice whatever of these invitations and we are getting it our way now to a certain extent. Now and then we have cases of a full-blown architect acting in a way that a decent fishmonger would be ashamed of, but the situation is improving and if you are far enough away to see it in its true perspective, you find it is getting very much better. The very fact that we are so shocked by the conduct of the gentleman who is a backslider is proof of this.

MR. BUTLER'S VIEWS

Following Mr. Brunner, Mr. Butler gave some details of the successive advances of the architects in dealing with competitions:

The result of our experience is that we must have an expert to run the competition, and the expert must be an architect. Some man must draw up the program and if an expert does it we will approve it.

By having an expert prepare the plan we are assured of fair play to all competitors, and that the winner of the competition shall be awarded the work. The latter point caused much difficulty, but we have finally taken the stand that no one will be allowed to enter the competition unless the owner is willing to give him the work if he wins.

VIEWS OF RALPH D. MERSHON

Ralph D. Mershon, past president of the American Institute of Electrical Engineers, presented briefly the engineer's point of view:

My position is this: I do not care to struggle against a condition which is economically sound. If the present condition is sound, there is no use in fighting against it, for it will overcome us eventually. If it is not sound, then fight it, but it is not clear to my mind that the present trend is not a sound one. I do not mean to say that it is a good thing for an electrical manufacturing company to design a plant and sell the machinery for it, because they will naturally recommend their own product whether or not it is the most suitable thing to meet the existing conditions. From that standpoint, it certainly is not to the advantage of the client or user that the design and construction should be wholly in the hands of the manufacturing company; it would certainly be better if they were in the hands of the consulting engineer, who is in position to select the best to meet the requirements of the special job.

On the other hand, in the case of this pioneer work which is done by the manufacturing companies and which the consulting engineer, in many cases, could not do for nothing, there is some question as to whether we should say that it should or should not be done by someone else.

JOHN F. WALLACE ON COMPETITION FROM MANUFACTURERS

John F. Wallace, president of Westinghouse, Church, Kerr & Company, in his brief remarks pointed out a change in the attitude of some large corporations with reference to engineering work done by manufacturers.

There have been two instances which

have shown the tendency of large corporations to employ other than electrical engineers to do their work. In one of these cases Mr. Gibbs was interested. In this case, however, the electrical engineer of one of the large corporations undertook, in an interview with the gentleman who employed Mr. Gibbs, to tell him that Mr. Gibbs was mistaken. In that case, Mr. Gibbs' client showed the manufacturer's representative the door very politely but very forcibly. In another case my own corporation was employed by a large railroad to make a report on the electrification of their line, in spite of the fact that two of the largest electrical companies offered their services to the railroad free. In this case a part of the line had already been electrified and the question for decision was whether it should be extended and, if so, whether the electricity should be generated by steam or water power. The whole subject was placed in our hands for thorough investigation and we decided in favor of extension but against the use of steam.

I know of another case where the president of a road found that the expense of a consulting engineer was too high and thought he could trust his own friends to decide what appeared to be a perfectly simple proposition. As it turned out, the case was not so simple as it appeared at first sight and the road found that it would have been much less expensive to have secured the services of a competent consulting engineer. However, I am glad to say that the general tendency now seems to be toward the employment of competent consulting engineers and putting the whole thing in their hands.

WHAT ARCHITECTS HAVE ACCOMPLISHED

Robert D. Kohn began by answering a question put by P. W. Henry as to the way members of the American Institute of Architects met the competition of those not members of the institute. He said that they actually did not meet the competition of the non-members, and explained their position at some length.

Whereas, he said, there are about 20,000 architects in the United States and our membership is only about 2000 or 2200, I think we have the best of the lot and consequently, although there are competitions held under improper programs, in which there are conditions of competition which the institute does not approve, States, municipalities and corporations are interested as soon as they learn that there is a group of men from whom they had expected to receive designs who will not enter such competition. Sometimes they say that they can get along without us, but very frequently we find that the program is completely changed to suit the conditions which we consider are essential. To attain this end we have had to assume an attitude of helpfulness, and the diagram of which Mr. Butler has spoken has been very helpful in that direction. When we had a large printed sheet to show what a man had to do they would look at it and say, "This is fearful and we will get along without it," but with this program, which is simple in form, we have no such trouble. We can have a committee appointed by each side who will help them change the program or make it conform with the conditions we require without cost.

Frequently our attention is called to the fact that, in the case of a small municipality, a certain number of experts are required on the jury and they are not empowered by law to engage such experts. We have arranged in these cases to employ a jury, which will serve without pay, when the city or corporation is not empowered to employ such a jury themselves and where otherwise the competition would fail of approval. That frequently works, but we have by no means entirely accomplished the desired end in all cases.

I do not think that competition is necessarily the best way to get the most desirable thing in all cases. In Government or municipal work it is probably necessary in order to avoid political influence and for that reason we have prepared the necessary machinery. We do not say that we intend to discourage competition, but as a matter of fact, most of us realize that it does not always result in the best design. It depends entirely upon the jury. I, for one, do not believe that any really great thing will ever come out of competition, as far as the artistic side of the question is concerned, but as far as its effectiveness is concerned, it can be made perfectly fair.

We have had to say to the lawyers, who are usually the hardest to convince, "What would you do if someone came to you and said, 'I have a case coming up in court next month, and I will get my witnesses together and ask six lawyers to prepare briefs of the case; then I will read them over and pay the one whom I select—if I like any of them'?". That is what the architects are called upon to do and what those outside of the institute are still doing. And that is what the engineers are doing.

RECENT ACTION OF A BRIDGE COMPANY

Because of its bearing on the subject the secretary at this point read the following letter from Ralph Modjeski:

"It is because I believe that the following incident may elicit a discussion and perhaps an action of the institute which would tend to discourage similar incidents in the future, that I have decided to bring it before the institute.

"The new Memphis Bridge will consist of a continuous and cantilever superstructure with the following lengths of spans from east to west: An anchor arm 186 ft. long, a cantilever span 790 ft. long, a fixed or anchorage span 621 ft. long, and a contilever span 604 ft. long. In addition thereto there is an independent deck span at the west end of the main bridge, which, however, does not really enter into the discussion.

"I have prepared designs for this structure, and under instructions from the company, have submitted bids for tenders to four bridge companies, which in my opinion are able to handle the work. Two of these companies declined to bid. Of the two others, one was considerably lower than the other, and therefore I recommended that its proposition be accepted.

"While the matter was being considered by the officials of the railways concerned, the unsuccessful, though powerful, bridge company approached those officials over my head, and submitted a sketch plan, or rather merely a diagram showing four independent spans of the same length as my design for the continuous structure, with a claim that it would be lighter and therefore cheaper than my design. They proposed to erect a 790-ft. independent span by the cantilever method. Any one familiar with bridge work will see the absurdity of that statement, but the powerful bridge company did not hesitate to make it in order to upset matters, and reopen them with the prospect of their finally getting the contract.

"It should be stated that fortunately the men representing the railroads are of large caliber and easily understood the situation. The contract was finally awarded to the original low bidder.

"It seems to me that even had the powerful bridge company been so fortunate as to discover an improved design which would save many thousands of dollars, the matter should have been brought to the attention of the consulting engineer, and probably included in their tender for the work, in which case sufficiently full designs should have been furnished to demonstrate fully the feasibility and economy of the scheme. Even then the propriety of such a course seems to me questionable.

"I should be glad to see this matter discussed with the view of ascertaining if anything can be done to protect the consulting engineer from such unbusinesslike and unscrupulous actions on the part of the bridge company."

NECESSITY FOR ACTION

The final speaker was Eugene W. Stern, secretary of the institute. He said:

One of the chief reasons for the organization of this institute was to define clearly the limitations of the work of the consulting engineer, and the rules of practice which should govern his work. This we have done in our "Code of Ethics and Schedule of Fees," probably the first comprehensive attempt by any body of engineers to define their relations to one another and the public. Our action has been very generously followed by various engineering societies, both at home and abroad. This in itself is a very hopeful indication that this institute has a right to exist, and, therefore, that its members have a right to exist. Another one of the principal objects for which this body was formed was to prevent the contractor from interfering with the work of the consulting engineer, and a number of the discussions we have had since our organization have dwelt specifically on this question.

Mr. Modjeski's letter read this evening relates the facts of a particularly flagrant and disreputable action by one of the great bridge companies, and all of us have had somewhat similar experiences with manufacturers or contractors.

Now, having agreed that we have a right to exist, we *must* take some positive action to protect our interests against the aggressions of our illicit competitors or we shall soon cease to exist; the manufacturers and contractors will gradually crowd us out of business. We must take action, and we must do so promptly. If we submit gracefully and quietly to present conditions, we are recreant to our duty.

I believe, gentlemen, that our strongest weapon is publicity. We must educate the public to understand our functions and our value to it, and we must not cease our efforts in this direction, but keep at it continually. We need not only the help of our friends of the technical press, but also the help of the great daily newspapers, for it is only these that reach the lay public. We should invite the editors of the representative New York dailies to our meetings, and thus in time spread a knowledge of our functions.

Comparisons of Systems of Flood Control

Fifth Paper in This Series—Relative Costs of Storage Reservoirs as Compared with Other Means of Preventing Damage Due to Floods

By H. A. PETTERSON
Civil and Hydraulic Engineer, San Francisco

THE purpose of this paper is not so much to present new data on costs as to suggest a new viewpoint for comparing relative costs. If, as is demonstrated by the fourth paper of this series, storage reservoirs can be utilized for flood prevention simultaneously with their use for various industrial purposes, it is obvious that the total cost for reservoir construction should not be charged to flood prevention alone. The percentage chargeable to flood prevention will vary in different sections of the country not only with varying conditions of runoff but also with the value of the impounded water for the various industrial purposes. It is plain that if nearly all of the impounded water is first used for power development, then, after leaving the power house, this water is reused for irrigation, the power interests and the irrigation interests should pay for a certain percentage of the reservoir cost.

In Table 4 of the fourth installment the reservoir storage for flood control, irrigation and power is 7,077,000 acre-ft., for irrigation and power the storage is 4,841,900 acre-ft. and the excess for flood control equals 2,235,100 acre-ft.; 2,235,100/7,077,000 = 31.5 per cent of the total reservoir capacity that is chargeable to flood control.

In the following discussion the writer has assumed that one-third of the total cost of reservoirs should be charged to flood control.

STORAGE RESERVOIRS

Tables 1 and 2 contain costs for constructed and proposed reservoirs in the United States. Records of reservoirs in foreign countries were purposely omitted, as conditions there differ widely from those in this country. These tables include a total of 131 reservoirs, with an aggregate storage of nearly 19,000,000 acre-ft., and a total estimated cost of about $166,000,000. It is believed that the average cost of such a system of reservoirs, constructed under widely varying conditions, will furnish a reliable criterion for the cost of an extensive project of reservoir construction.

Table 3 contains an analysis of the estimated cost of some of the projected reservoirs of Table 2. It divides the total cost into four principal items and shows the percentage which each item is to the total cost. The average cost per acre-foot of storage capacity of all the combined reservoirs of Tables 1 and 2 is $8.88. One-third of $8.88 is $2.96, or, say, $3 as the proportion chargeable to flood control. If the total cost of $8.88 per acre-foot is used with the items and percentages of Table 3, the following itemized costs are the result:

Items	Percentage	Cost per acre-foot
Dam and appurtenances	54	$4.80
Right-of-way, including buildings and clearing	16	1.41*
Damages to railroads, highways, etc.	17.4	1.55
Engineering and contingencies	12.6	1.12
Total	100	$8.88

*$1.41 per acre-foot is equal to $56.40 per acre for mean depth of 40 ft. The California Débris Commission used an average of $33.35 per acre for high-priced lands of the Sacramento Valley in its estimate of the By-Pass.

The estimated cost of the By-Pass project in the Sacramento Basin, as compiled from the 1911 reports of the California Débris Commission in House Document 81, Sixty-second Congress, First Session, is as follows:

Estimated cost of By-Pass	$32,389,090
Expended by the State on work along river	2,500,000
Estimated expenditure by 41 of 48 reclamation districts	16,801,578
Subtotal	$51,690,668
Bank protection, 200 miles	10,000,000
Approximate grand total	$61,690,000

No allowance for revetment is included in the commission's estimate. Perhaps no engineer will question the necessity for bank protection when all conditions of the lower Sacramento are taken into account.

Annual Cost

Interest at 5 per cent	$3,084,500
Maintenance and renewals on levees, 5 per cent	2,584,500
Maintenance and renewals on revetment, 10 per cent	1,000,000
Total	$6,669,000

It has been shown that $3 per acre-foot is the estimated cost of reservoir construction chargeable to flood control. On this basis an expenditure of $61,690,000 would suffice for the construction of 20,563,000 acre-ft. of reservoir storage capacity, which probably is three times greater than would be required. A comparison on the basis of annual costs puts reservoirs in a still more favorable light, as the maintenance charge for reservoirs is low. The annual charges are assumed as follows:

	Per cent
Annual interest	5
Maintenance and depreciation	1
Total	6

The annual cost for the present scheme was shown to be approximately $6,669,000, which, capitalized at 6 per cent (the percentage per annum for reservoirs), equals $111,150,000. This sum would be the allowable expenditure for reservoirs for flood prevention alone.

Objections might be made on the grounds that some of the expenditures included in the estimate of the By-Pass system would also be required for the reservoir system. The annual cost for dredging and maintenance of a navigable channel would be necessary in either case. This item, however, is not included in the By-Pass estimate and can, therefore, be omitted from the comparison. The portion chargeable to reservoirs is the enlargement of the channel capacity from the mouth of the Feather River to the mouth of Cache Slough. This, in the commission's estimate, is only a small percentage of the total cost. The items for bank protection and levees would not be required, and local drainage of the swamp lands could probably be secured without the use of pumping stations, with their high first cost and maintenance charges. It is obvious that those parts of the commission's estimate which should also be charged to reservoirs are balanced by other items omitted from the By-Pass system.

A conservative estimate of the storage capacity in reservoirs for the Sacramento watershed is 10,000,000 acre-ft. This is for flood prevention combined with complete regulation for power, irrigation, water supply and navigation. On the basis of $3 per acre-foot the total cost chargeable to flood prevention would be $30,000,000, which is less than one-half of the estimated total of the By-Pass system.

MISSISSIPPI BASIN

House Document 638, Sixty-second Congress, Second Session, contains an estimate of the revetment required and the cost thereof from Cairo, Ill., to the head of the passes, prepared by the Mississippi River Commission for the information of the Congress. The following is an extract from the report:

"The Mississippi River Commission has caused examinations and surveys to be made at various times to determine the extent of caving on the Mississippi River below Cairo and the amount of revetment that is necessary for the proper protection of its banks. These surveys indicate that during the period from 1882 to 1910 caving has occurred on one bank or the other over the entire distance from Cairo to the mouth of Red River. Below Red River caving is less active, but is estimated by the officer in charge of the improvement of the fourth district that 28 miles of bank on this section of the river now require revetting, and for complete treatment this length should be increased to 40 miles.

"A survey of caving banks from Cairo to the mouth of Red River made under the supervision of the secretary of the commission during low water in 1907 shows the total length of banks then actively caving to have been 749 miles over a channel length of about 790 miles. . . . The cost of the work has gradually increased, as it has been found necessary to thicken and widen the channel mats as well as to exercise greater care in grading and paving the upper bank slope, and is at present estimated at $200,000 per mile, giving an estimated cost of $157,800,000 for the revetting of the banks necessary for the complete treatment of the river from Cairo to the head of the Passes.

"The length of bank line previously revetted has not been included in the above estimate."

This estimate does not include any allowance for the navigable tributaries. Based on a ratio of the expenditures on the lower Mississippi to the total expenditures, the allowance for revetment should be doubled —that is, for the lower Mississippi the estimate equals $150,800,000; for the tributaries of the entire Mississippi, $150,800,000, making a total cost of revetment of $315,600,000.

The maintenance and renewals for bank

LEVEES

Total yardage in a levee system for the entire Mississippi basin (see second paper, page 427) equals, cubic yards	1,107,500,000
Cost at 25 cents per cubic yard	$276,875,000.00
Spent on levee construction to June 30, 1910	82,891,351.77
Estimated cost for completion of levee system	57,000,000.00
Total for 443,000,000 cu. yd. of adopted levee line of lower Mississippi	$139,891,351.77

Average per cubic yard for levees is 31.6 cents.

The approximate cost for flood protection on the Mississippi River and tributaries would be:

For revetment	$315,600,000
For levees	276,875,000
Total	$592,475,000

TABLE 1—DATA ON CONSTRUCTED RESERVOIRS

Location and name	No.	Capacity, acre-ft.	Type of dam	Costs — Total	Costs — Cost per acre-ft.
Haiwee, Cal.	1	63,800	Hydraulic fill	$432,080	$6.78
Olive Bridge, N. Y.	1	368,000	Masonry and earth	26,000,000	70.70
New Croton, N. Y.	1	98,300	Masonry	8,000,000	81.30
Wachusett, Mass.	1	194,200	Masonry	6,500,000	33.40
Reservoirs built by U. S. R. S. (a)	10	3,799,300	Various	10,005,000	2.63
Upper Mississippi (Lakes)	5	2,100,000	Timber and crib dams	750,000	0.36
Indian River, N. Y.	1	115,000	Masonry	120,000	1.04
California (b)	12	325,036	Various	2,063,480	6.35
New Mexico (c)	2	95,300	Rock fill and earth	356,000	3.74
Colorado (d)	8	42,809	Earth	377,592	8.80
Totals	42	7,101,745		$54,604,152	7.69
Proposed reservoirs	89	11,671,548		111,823,854	9.58
Grand total	131	18,773,293		$166,428,006	$8.88

(a) Roosevelt, Ariz.; East Park, Cal.; Deerflat, Idaho; Carlsbad, N. M.; Cold Springs and Klamath, Ore.; Belle Fourche, S. D.; Bumping Lake, Wash.; Pathfinder and Shoshone, Wash.

(b) Sweetwater, Bear Valley, Hemet, Escondido, LaMesa, Cuyamaca, Buena Vista Lake, English, Bowman, San Leandro, Eureka Lake, Fanchine.

(c) Lake Avalon and McMillan, Pecos River, N. M.

(d) Cache La Poudre, Larimer and Weld, Windsor, Monument, Apistrapar, Hardscrabble, Boss Lake and Saguache.

protection on the Missouri River equal 22 per cent per annum of the first cost, but in spite of this the allowance of 10 per cent has frequently been used in estimates by the army engineers. The maintenance of levees, including repairs from breaks, work during high waters, etc., is taken at 5 per cent annually.

TABLE 2—DATA ON PROPOSED RESERVOIRS

Location and name	Number	Capacity acre-ft.	Type of dam	Cost per acre-ft.
Allegheny Basin (a)	21	1,142,000	Concrete overflow	$18.82
Monongahela Basin	22	706,500	Concrete overflow	17.90
New York, Sacandaga	1	852,000	Masonry	5.35
New York, Portage	1	585,000	Masonry	8.63
California, Long Valley	1	341,000	Concrete	2.48
California, Sacramento Basin (b)	12	2,388,100	Various	3.76
California, Clear Lake	1	100,000	Regulating weir	4.52
N. Y. State (c)	9	2,425,550	....	17.62
Passaic River & Mountain View Reservoir	1	176,200	Emb. and core wall.	21.82
Arizona (d)	5	853,336	Masonry and rock-fill	8.06
California (e)	2	536,400	Masonry and rock-fill	1.06
Rock Creek, Nev.	1	80,000	Rockfill	1.00
Columbus, Ohio	1	17,440	Masonry	18.60
New Mexico (f)	2	453,368	Masonry and rock-fill	0.95
Sand Lake, Texas	1	72,500	Natural basin	0.50
Wyoming (g)	5	826,413	Various	2.31
Colorado (h)	3	115,741	Various	7.92
Totals	89	11,671,548		$9.58

(a) See Table 33 of Report of "Pittsburg Flood Commission."

(b) Guenoc, Monticello, East Park Site, Briscoe, Jess Valley, West Valley, Warm Springs, Valley, Mill Site, Ash, Round Valley, Big Valley, Iron Canyon and Grizzly Valley.

(c) Schrom, Tumble Falls, Black River, Salmon River, Piesco Lake, Stormville, Billings, Ashokan, Hibernia, and Silvernails.

(d) San Carlos, Riverside, Buttes, Horseshoe, and Bear Canyon, Ariz.

(e) Victor and Wanache Meadows, Cal.

(f) Elephant Buttes and Pecos River, Nev.

(g) Laramie, Sweetwater, Cloud Peak, Piney, and Lake Devinet, Wyo.

(h) Loveland, Tarryall, and Los Canyon, Colo.

Estimates include all structures, damages, relocations of highways and railroads, etc.

The cost of storage reservoirs per acre-foot chargeable to flood prevention will be taken at $3 as before. On this basis a total of 197,491,666 acre-ft. of reservoir storage could be constructed, or at least twice as great a capacity as is required.

COMPARISON ON BASIS OF MAINTENANCE

The total annual charge for interest, depreciation and maintenance of reservoirs equals 6 per cent as before. The annual maintenance charge for levees and revetment equals $75,027,500, which, capitalized at 6 per cent, equals $1,250,458,300. This amount expended in reservoirs for flood prevention only would provide for a storage capacity of about 417,000,000 acre-ft., or at least four or five times the maximum that will ever be required for the entire Mississippi watershed.

The comparative costs presented here are, of course, approximate. The allowances for the numerous tributaries are merely an estimate based on the published estimates of the lower Mississippi. However, when the estimated and actual costs of flood prevention are added for cities like Pittsburgh, Pa., Columbus, Dayton, Hamilton and Middletown, Ohio, and Topeka and Kansas City, it is found that the expenditures for these cities alone will amount to $100,000,000. The estimate is believed to be extremely fair to the levee system. It omits all reference to drainage by pumping neccessary by construction of levees, to the heavy charges for the installation of these numerous pumping plants and the cost of operating and maintaining them, nor does it add anything for cutoffs of sheet piling, which in many places is proving necessary for the stability of the levees; nor is any allowance made for a concrete lining on the river face of the levees to protect them against wave wash and burrowing animals. Many engineers believe that a long line of levees will never be safe from breaks until the cutoffs of sheet piling and the facing of concrete are employed throughout the entire length.

TOTAL COST PER ANNUM OF LEVEE SYSTEM

Interest 5 per cent of $592,475,000	$29,623,750
Maintenance of revetment, 10 per cent of $315,600,000	31,560,000
Maintenance of levees, 5 per cent of $276,875,000	13,843,750
Total	$75,027,500

CONCLUSIONS

The writer does not pretend to state that reservoir control is practicable for the Mississippi. He can state with positive assurance, however, that no one else, not even the members of the Mississippi River Commission, can disprove the practicability of reservoirs and furnish convincing proof. The data to prove or disprove the case are not available, because the necessary engineering surveys of various kinds have not been made. There has never been a thorough investigation of the subject. All previous decisions of engineering boards have been based on opinions.

The figures presented in this paper are approximations, but they show that reservoirs could cost considerably more, or levees and their attendant construction cost considerably less than the figures compiled, and still the reservoir system would be worthy of a real engineering investigation.

When the costs, by whatever system executed, are going to run into hundreds of millions it would seem the part of prudence to thoroughly investigate all possible solutions.

TABLE 3—ANALYSIS OF ESTIMATED COSTS

Location and name	No. of basins	Capacity, acre-ft.	Acres submeged	Type of dam	Dam and appurtenances	Right-of-way, including buildings and clearing	Damages—railroads, highways, etc.	Engineering and contingencies	Total	Cost per acre-ft.
Allegheny Basin	21	1,142,000	26,826	Concrete overflow	$11,336,200	$3,140,300	$4,246,900	$2,808,500	$21,531,900	$18.82
Monongahela Basin	22	706,500	17,094	Concrete overflow	9,026,800	774,300	1,189,200	1,648,600	12,638,900	17.90
N. Y. Sacandaga	1	852,000	24,650	Masonry	1,061,000	2,282,000	603,000	604,000	4,550,000	5.35
N. Y. Portage	1	585,000	8,770	Masonry	2,091,000	775,000	1,720,000	458,000	5,044.000	8.63
Calif. Long Valley	1	341,000	8,150	Concrete	560,000	204,000	....	84.000	848,000	2.49
Totals	46	3,626,500	85,490		$24,075,000	$7,175,600	$7,759,100	$5,603,100	$44,612,800	$12.30
Per acre-foot	..	1	0.0236		$6.63	$1.98	$2.14	$1.55	$12.30	......
Percentage of total cost	..	....	......		54	16	17.4	12.6	100	......

Open Caisson Difficulties

OPEN steel cylindrical pier caissons for the Newberg Bridge, described in the Engineering Record of April 11, page 420, were sunk partly by interior dredging and partly by interior hand excavation. In one case hard clay bottom, encountered 12 ft. below the surface of the water, could not be handled by an orange peel bucket, and the excavation was continued about 40 ft. farther by hand, after the caisson was pumped out by a 6-in. Emerson pump. At first great difficulty was occasioned by water leaking through the cylinders or from pipes, and generating so much steam by contact with the hot pump casing that the workmen were seriously hampered by it. Finally a hose line was run from the air compressor to the bottom of the excavation, and blew out the steam sufficiently to keep the air clear.

Several logs 3 ft. in diameter were encountered from 10 to 20 ft. below the river bottom, and were cut through by dropping on them a weighted chisel operated by a derrick boom. The chisel was made of a 6 x 1-in. steel plate 8 in. long with a 6-in. sharpened cutting edge. It was bolted to a pair of 60-lb. rails 30 ft. long, braced together and swung from the boom. A man in the cylinder guided the chisel and signaled the engineman who raised the chisel 5 ft. and dropped it as fast as possible. After the logs were cut out inside the cylinder, the chisel was reset at an angle of 17 deg. with the vertical, and the ends of the logs were trimmed off underneath the edge of the cylinder.

Deep Sewer Work at Minot, North Dakota

Abstract of Paper Read at Annual Meeting of North Dakota Society of Engineers Feb. 12 by J. R. Graham, Assistant City Engineer of Minot

INSTALLING 3900 lin. ft. of 12-in. tile sewer pipe below the ground-water level in unstable alluvial material is a difficult feature of the sewer improvement in Minot, N. Dak., a city of about 6200 population. The improvement, which is near completion, also includes 15,781 ft. of 10-in. shallow sewer, 11 deep manholes, 46 shallow manholes and a concrete wall 8½ ft. wide and 100 ft. long to carry water and sewer mains across the Mouse River. The original contract prices were as follows: River crossing, $2800; deep manholes, $150 each; shallow manholes, $50 each; 10-in. sewer, 75 cents per linear foot; and 12-in. sewer, from $4 to $6 per linear foot. Including extras, the cost will total about $40,500.

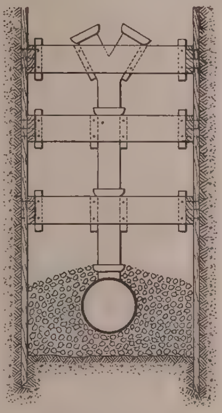
Two residence districts lie on either side of the river in the west end of the valley section of Minot, extending 2 miles upstream from the outfall. The main trunk line accommodates one-half of this section; the sewage of the remaining half will be carried by the deep sewers under discussion, which connect with a pumping station at the trunk intersection. The sandy soil contains a little loam—not enough to allow of the banks standing without sheeting. At indefinite intervals blue clay is encountered. All ditches have to be well braced as soon as dug, as the air seems to slack the soil, causing caves.

Starting April 18, 1913, a crew of from 6 to 15 men worked five weeks on 50 lin. ft. of ditch, but were unable to lay a single pipe. Sand points failed because too much loam was present; wood sheeting proved insufficient and, though steel sheeting was tried, the contractor decided to sublet the contract to avoid investing in new machinery.

On July 20 the subcontractor began work with an equipment consisting of a Parsons excavator, a two-bucket Potter machine, a 4-in. centrifugal pump with a 6-hp engine and boiler, a 5-in. centrifugal pump with a 10-hp engine and boiler, three hand pumps and four teams, besides the crew. The Parsons machine was used in front of the Potter to excavate a ditch 5 ft. wide to a depth of about 9 ft., where water was usually struck. Temporary braces were put in about 8 ft. apart, which were removed by the sheeting crew behind the Potter as they set the long sheeting.

Our work was ideal for the Potter machine, which is most economical on long stretches of straight ditch, since it takes a crew of eight or ten men about six days to take it down or set it up again. The two main stretches of our work were long ones—1700 and 1800 ft. The machine was usually moved at intervals of 64 to 72 ft., as a section of this length was found to be most economical to work.

First in the section of ditch that was being worked came the sheeting crew. This consisted of two men trimming banks and setting sheeting in place and from eight to ten men driving with wooden mauls. The sheeting was 2 x 8-in. yellow pine lumber in 18-ft. lengths. In the 20-ft. work stub sheeting—fir in 10-ft. lengths—was used in the bottom and had to be left in. A car of yellow pine was obtained at the beginning of the job last September, and all, except what was broken in pulling, is still in use, apparently as good as ever. The ends do not batter down so badly when driven as fir and it does not break or split easily. If clay was found the sheeting was driven about 1 ft. below grade, but if the material was sand — which was usually the case—2 to 3 ft. was none too deep.



House Connection, Showing Bracing

Bracing and Drainage

For horizontal bracing four sets of 4 x 12-in. stringers 16 ft. long were used. The first set was put in about 4 ft. from the top of the bank and the others were placed 4 ft. apart as the ditch was dug. These stringers were braced with 3-in. iron jacks, set 8 ft. apart, and every 16 ft. where the ends of the stringers come together two jacks were used. Thus the ditch was divided into 8-ft. sections, affording ample room in which to load a bucket. Six sets of buckets kept the machine busy.

The water was taken care of by making a sump 18 in. square and 18 in. deep in the first manhole built on a line. By the use of the 4-in. pump and the 6-hp engine the water allowed to run through the constructed line was pumped out and run through troughs into the river, a short distance away. In daytime the pump had to be run at intervals of fifteen or twenty minutes, and it required about three minutes to pump out the accumulated water. At night it was not necessary to keep the water so low, but it was never allowed to rise higher than the top of the last pipe laid, for if, because of a breakdown of the pump, the water rose in the ditch the banks would become saturated and sand would run through any small crack in the sheeting or would boil up in springs from under the sheeting. If these springs were permitted to run long the banks would settle and stringers begin to crack, and the muck in the ditch would become so water soaked that men working in it would have difficulty pulling themselves out of holes. If the water was kept down it drained the soil for a distance of about 30 ft., and dry digging was had down to within about 2½ ft. of grade.

Heavy Bracing in Deep Sewer Trench

Another effect of the water rising in the ditch was to raise the pipe that had been laid. At one time when the pump had broken down the water rose 4 ft. It was subsequently found that 36 ft. of pipe which had been inclosed with gravel and concrete had been raised from 1 to 3½ in. above grade. This was undoubtedly due to the water-soaked banks settling and raising the bottom of the trench. The remainder of the day was spent in re-laying them, and all pipes were thereafter braced by means of 2 x 8-in timber resting on the top of the pipe and wedged under a jack above.

Gravel Foundation

One of the most important requisites was a solid foundation for the pipe. This was accomplished by digging 6 in. below grade and filling in with coarse gravel, which was allowed to stand three or four hours before pipe was laid. The sand filled in the voids of the gravel and the mass would become almost as solid as concrete. Gravel was also used to advantage in stopping springs in the bottom. Soon after applying a few buckets of coarse gravel, a hole into which a man would sink to his waist would present a hard, watertight bottom. The gravel seemed to act as a binder for the sand, and without it the work would have been almost impossible. We were very fortunate in having an abundance of gravel within half a mile of the work.

The pipes were in 3-ft. lengths, with extra-large bells, which made it easy to get the oakum in and run the asphalt into the joints, to make which, in all of our deep work, we used Pioneer asphalt or Sarco. Two pipes were put together on top by setting them on end one on top of the other, pushing a string of oakum into the bell with a yarning iron and pouring the bell full of the hot asphalt, which was allowed to cool thoroughly before being lowered into the ditch.

Most companies advertising sewer jointing compounds claim they can run joints in water. In order to get a good joint we have found it necessary to have the joint free from water. In every case where water was up above the bottom of the bell a small space was left in the bottom where the jointing compound did not come together, due to cooling by the water. We seldom fail to get a good joint when the water is down and the compound is hot enough.

In laying pipe in a wet trench the double joint that has been prepared on top is let down by the cable on the carriage of the Potter machine and the pipe put in place by the pipe layer and his helper, who are aided by use of a wire sling which is placed around the pipe near the spigot end and a hand line hooked into this, by means of which two men on top raise the pipe while the pipe layer fits the oakum. The helper then pushes the pipe home, and after it has been brought to line and grade a dam is built to the top of the pipe at the middle joint with a couple of buckets of concrete and extra gravel. This shuts the water off from the joint and forces it through the pipe. The water is then dipped from around the joint, the oakum thoroughly calked in and the asbestos roll or runner placed around the joint. When all is in readiness the hot asphalt is let down in a 1-gal. bucket and the joint is poured. Dipping is then stopped and the water allowed to rise on the joint and, to hasten the cooling further, water is dipped from the trench and poured on the joint. In about three minutes it is cool enough to allow taking off the roll. The water is again dipped out, the joint examined and, if good, concrete is placed around and tamped underneath it. Concrete is also put under the invert up to the springing line along the entire length of pipe. This is done as a safeguard against settlement and also to reinforce the pipe. The concrete is covered with gravel to protect it from being washed by the water.

Another reason for placing concrete around the joint is to seal the asphalt in the joint. There will be times when repairs will have to be made at the sewage-pumping station and the sewage will be allowed to back up in the pipe until there will be a head of 10 ft., under which a weak place in a joint might easily become a leak. Thus far our work has proved tight. The cost of the concrete around the pipe is about 10 cents per foot.

BRACING FOR HOUSE CONNECTIONS

All house connections have been built up to from 5 to 10 ft. above the invert of the main sewer. We have used T's instead of Y's in order that the part built up might be braced more easily and more securely. Concrete was placed around the 12-in. T and built up to the bell on the 6-in. opening. This was for the purpose of reinforcing the 12-in. pipe to enable it to withstand any extra weight due to the built-up section. On top of each of the connections a breech was used, and in this way two houses were served by one connection, thereby reducing our built-up connections one-half.

The original contract was awarded to J. M. Carroll & Son, of Grand Forks, N. Dak. The deep-sewer work was relet to James Kennedy. At the time the address was delivered there remained to be completed 1000 ft. of the 15,781 ft. of shallow sewer, 500 ft. of the 3900 ft. of deep sewer and practically the entire concrete river crossing.

Philadelphia Transit Construction

Typical Designs for New Subway and Elevated Railroad Structures to Cost $57,578,000

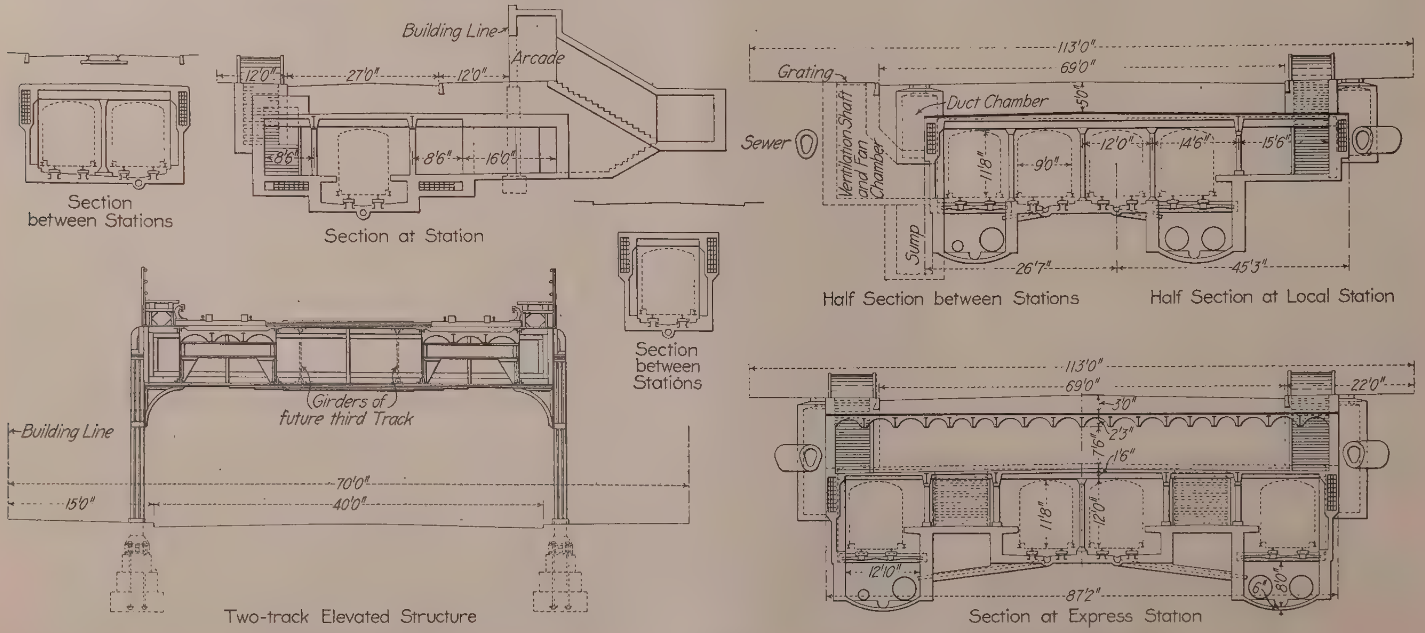
THE new transit system recommended by Commissioner A. Merritt Taylor for the city of Philadelphia involves the immediate construction of 8.6 miles of subway and 16.9 miles of two and four-track elevated railroad structures, at an estimated cost of $57,578,000, as described in the Engineering Record of March 14, page 308. It will provide for the transportation of 206,000 passengers per hour and is estimated to serve 60 per cent of the population of 1,623,200 people in the district affected. The estimated cost of the two-track subway without equipment is about $1,000,000 per mile, while that of the two-track elevated structure with solid floor is about $400,000 per mile. The typical structures recommended vary materially from those generally built for such purposes and are here illustrated.

The system is planned for the operation of ten-car trains on 90 seconds headway, making a capacity of 400 cars per hour per track, which will correspond to a total capacity of 206,000 passengers for the system.

The subway structure is made entirely of steel and concrete, with a structural steel framework for roof and walls. The flat roof is waterproofed with asphalt mastic and, except where special waterproofing is used for unusually heavy hydrostatic pressure, the use of dense concrete in the sides and bottom is expected to give sufficient impermeability to external water. The general features involve a rectangular cross-section with continuous full-height longitudinal partitions separating the tracks. The roof over each track will be arched and the track rails will be laid on short transverse wooden ties seated on the concrete floor without ballast. The over-all width of the four-track regular structure is 53 ft. 2 in. The height from base of rail to top of roof is about 14½ ft., and the minimum cover over the roof is 5 ft. The standard minima adopted include 165-ft. track radius, 12 ft. spacing of tracks, and 12-ft. headroom clearance. Express tracks are located in the center, and local tracks on the outside of the subway, and the latter have full width deep chambers under them forming galleries for the installation of pipes. Duct chambers are built into the upper portions of the outside walls, and at intervals, openings in the outer walls connect with ventilation shafts terminating in sidewalk gratings.

A portion of the system known as the Delivery Loop subway has both single and double track structures in narrow streets. The single track structure has an inside width of 10 ft. 8 in. and a clearance height of 12 ft., and the double-track structure has a clear width of 9 ft. for each track Both of these types are drained by a single sewer built integral with the bottom of the structure.

The profile of the Broad Street subway is broken on account of the effort to keep the stations and station platforms as near the surface as possible, and the necessity of passing under many existing sewers and other structures. This construction was necessitated by the existence of a number of main sewers up to 18 ft. in diameter, which cross Broad Street. Leakage into the subway is collected in drains or gutters delivering to sumps emptied by electrically driven pumps. The ventilation of the subway is planned so that it may be benefited by the plunger action of the trains, and may hereafter be modified by the installation of exhaust fans, and it is hoped that in the new buildings along the subway lines ventilation shafts will be provided to discharge the heated air above sidewalk level.



Typical Sections of Two and Four-Track Subways and Stations and Elevated Railroad Bent

Broad Street occupies the summit of the land separating the Delaware and Schuylkill Rivers, and the soil consists principally of sand and gravel. Top soils and clay are usually encountered on the surface, good gravel is reached 15 or 20 ft. below it, and at a depth of about 40 ft. is cemented hard and has great supporting power. Pockets of quicksand are occasionally encountered and rock is reached a few feet below the cement gravel. Preliminary borings about 500 ft. apart indicate that not much rock will be encountered in the South Broad Street or loop subways but that near Pike Street the excavation will be almost entirely in rock.

Except in a few locations it is expected that the excavation will be made by open cut methods similar to those used for the New York subway. Here one-half of the street will be planked and kept open for traffic, while the other half will be closed to traffic, thus facilitating more rapid construction. Electric power will be required for hoisting and pumping, and the work will be divided into contract sections of such a length as may be completed in from two and one-half to three years each. The excavation will amount to at least 1,000,000 yards and will, it is estimated, be handled by teams and two-yard dump wagons for short hauls, and by four to six-yard motor trucks with or without trailers for long hauls. The spoil is valued at 20 cents per yard for use in park and street fills.

STATIONS

The subway stations are planned to accommodate trains of ten cars 50 ft. long, each having six side doors operated by compressed air so that the maximum and average station stops will not exceed forty and twenty seconds respectively. The cars will weigh 40 tons each, and will be equipped with two 125-hp motors each, and an electrically operated automatic air brake system. Several different types of subway stations are proposed for single-track, double-track, four-track, and for local and express service. On account of the narrowness of the streets in the business districts, where the single and double-track loop is located, a special design of station entrance has been devised. The front of the building is portalled, forming an entrance lobby of the full height of the first story, inside the building line, which gives access to the subway stairs without diminishing the sidewalk width.

Express stations are made with long platforms 20 ft. wide, between the express and local tracks, which are connected at both ends with stairways to wide mezzanine floors below street level, having entrances and exits on both sides of the street. Local stations are made with separate platforms on the outside of the local tracks, with two exits, one near the quarter point of each platform, and two entrances at the center of each platform. For the express stations at very busy points it is suggested that an additional exit platform may be built between the express tracks, to permit passengers to reach the street without conflict with transferring passengers.

Studies have been made for special stations designed for alternate trains to use alternate sides of the station and thus enable them to be run on shorter headway without danger of collision with trains standing at stations. Theoretically this system increases the station capacity from 50 to 75 per cent, but on account of the excessive width required its use is not contemplated in the subway.

ELEVATED RAILROAD

The elevated railroad structure consists of longitudinal plate girders web-connected to transverse plate girders of the same depth, supported by vertical columns on the curb lines. The transverse girders are heavily knee-braced to the columns and are also connected to the latter through flange rivets as well as by their drop ends seated on the column tops. The ballasted track is laid on a concrete floor supported by I-beam stringers carried on transverse floor beams at about the middle of the longitudinal girders. The width of the structure varies according to the width of the street, from 34 to 40 and 44 ft. between curbs. In the narrowest width the footwalk cantilevers beyond the column centers, but in the two wider structures it is supported directly on the top flanges of the transverse girders.

Sanitary Statistics for Michigan

Survey of Treatment Costs and Relation of Typhoid Death Rate to Water Supply, Size of Community and Density of Population

VITAL statistics for Michigan, in relation to type of water supply and size and degree of congestion of the district served, in addition to costs of water purification and sewage treatment, were included in an address by Prof. W. C. Hoad, of the University of Michigan, at the annual meeting of the Michigan Engineering Society, which was held in Lansing Jan. 13-15. The following notes have been abstracted from the paper, which dealt primarily with the Michigan Water and Sewage Law and the Grand Rapids stream pollution decision.

A comparison by the Board of Health of the annual death rates from 1901 to 1910 inclusive of all Michigan cities of more than 5000 population showed that there were twenty-two surface supplies and nineteen well supplies. The mean annual typhoid death rate for the former class was 53 per 100,000, and that for the latter 24.6 per 100,000. That is, the rate for cities with surface supplies was 115 per cent higher than that for cities with ground-water supplies. However, in several instances filtration or hypochlorite treatment has since been introduced, reducing the rate for the former.

TYPHOID MORTALITY

The results of a more recent study are exhibited in the accompanying table, compiled from reports of the Board of Health, showing the mean annual typhoid death rates in cities and villages in Michigan, arranged according to size.

The table shows that, for the last five-year period for which complete records are at this time available, the mean annual typhoid death rate, contrary to the usual rule, was 64 per cent higher in cities than in the country. If the typhoid death rate in the cities could have been reduced to the standard of the rural rate it would have resulted in approximately 178 fewer deaths per year, or a total saving of 890 lives during the period covered by the table.

With respect to the distribution of the urban death rate according to groups, the table shows that the medium-sized cities—those ranging from a population of 5000 to a population of 50,000—exhibit a very much higher rate than either the small group of cities having populations over 50,000 (in which the city of Detroit exerts a preponderating influence) or the very large group of cities and villages having populations under 5000. This is true particularly of those cities ranging from 10,000 to 50,000, undoubtedly due largely to the fact that many of these cities take their water supplies from sources that have gradually become more and more polluted, or because of increased demands for water have been forced to develop supplies from surface sources in addition to or in place of their former ground water supplies, and that in these changes the purification of the supply has not kept pace with its increasing pollution.

The pollution of the streams of the State is steadily increasing. There are at present on the drainage area of Grand River twenty-two cities 1000 or more in population, with a combined population of 224,900 (census of 1910). This is a city population of about 40 per square mile of drainage area. On the Saginaw River drainage area are twenty-three such cities, with a combined population of 182,600, or a city population of 28 per square mile. Similar figures for Kalamazoo River show a city population of 41 per square mile of drainage area. Certainly by the time such figures as these are reached in the case of streams used as sources of public water supply some positive and constructive action should be taken, first, to provide thoroughgoing purification of the water previous to pumping it into the mains, and, second, to set limits to the pollution of the streams from which the water supplies are derived.

MEAN ANNUAL TYPHOID DEATH RATES IN MICHIGAN CITIES, VILLAGES AND RURAL DISTRICTS, 1908-1912 INCLUSIVE

Type of community	Total population in 1910	Typhoid death rate
Cities more than 50,000	628,847	23.7
Cities of 25,000 to 50,000	211,082	34.2
Cities of 10,000 to 25,000 and Calumet Township	223,916	42.2
Cities and villages of 5000 to 10,000	166,749	27.4
Cities and villages under 5000.	402,408	23.3
Total urban	1,633,002	28.0
Rural districts, exclusive of Calumet Township	1,177,171	17.1
Total and average	2,810,173	23.3

COSTS OF WATER AND SEWAGE TREATMENT

Complete water purification, including sedimentation with the help of coagulant, filtration, and final treatment with hypochlorite, will ordinarily cost about $10 per 1,000,000 gal. treated, or from about 30 to 40 cents per capita per year. This figure includes not only operating and ordinary maintenance expenses, but interest on the initial cost of the plant and a sinking fund allowance sufficient to replace the plant at the end of its useful life.

Obviously no very satisfactory figures can be given for the cost of sewage treatment, as the expense will depend largely upon local conditions. In a general way, however, it may be stated that in the case of a city having an efficient system of sanitary sewers from which all storm water and practically all ground water are excluded, the first cost of a plant consisting of septic tanks or two-story tanks for the preliminary process and broken stone or gravel or cinder contact filters for the finishing process may be taken at about $40,000 for each 1,000,000 gal. daily capacity, or about $4 per capita for the population served by the plant. This does not include any intercepting sewer, pumping plant, or long outfall sewer that may be necessary. The total yearly cost, including expenses of operation and maintenance, interest on first cost, and depreciation, may be taken roughly at from $12 to $15 per 1,000,000 gal. treated, or about 30 cents per year per capita of tributary population. If it is necessary to pump the sewage, this service will usually increase the per capita cost to 75 cents or $1 per year.

These figures for the cost of sewage disposal are much higher than those frequently advanced. In fact, communities have often been permitted to believe that the first cost is the only cost, unless the sewage has to be pumped. It is true that the amounts spent in routine operation are very low for many disposal plants operating wholly by gravity and controlled by automatic regulating devices, particularly during the first few years following their installation, when their loads are light. But the total annual cost, counting operating expense, interest on capital invested and depreciation, for each person tributary will not often fall much below the figure mentioned.

RÉSUMÉ OF SANITARY SITUATION

The pollution of the waters of Michigan is steadily increasing. The rapid, shallow flow over sandy beds, the relatively high ratio of minimum to mean discharge, the absence of the effect of pools at times of minimum flow, so common a feature of streams in the plains country—all these characteristics of typical Michigan streams, while excellently adapted to the prevention of local nuisances at the sewer outlets, are factors which greatly increase the distance to which the sewage menace will reach. The situation calls for a broad, constructive policy inaugurated and carried out in the interest of the public welfare. Such a policy seems to be outlined in the Water and Sewage Law and in the Grand Rapids decision, both of which were promulgated last year.

Bridge from India to Ceylon

A RECENT survey has demonstrated clearly what was previously considered probable—the practicability of constructing a bridge on the rocky reef called Adam's Bridge, which connects the island of Ceylon with the main land of India. The distance is 22 miles, and an estimate has been made giving the cost of the work, which probably includes a good deal of causeway, at about $4,000,000—an amount much less than was previously thought necessary. There is, therefore, a probability of this important connection being carried out in the near future. A part of it, from the main land to the island of Rameshwaram, has in fact been built. The immediate utility of such a bridge is much diminished by the fact that while the main-line railways are uniformly of 5-ft. 6-in. gage, the gage of the Adam's Bridge branch is narrower. Hence the excellent harbor at Colombo would still lack suitable connection with the rest of India.

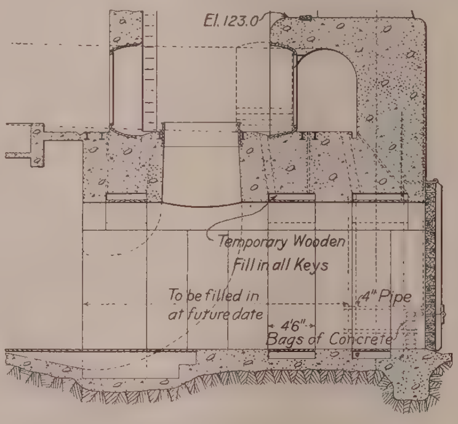
Brunet Falls Dam Closures

Methods Employed in Closing under 53-Foot Head Temporary Passages Left through the Foundations of the Power House

By ARTHUR R. RHENISCH
Former Resident Engineer, Brunet Falls Manufacturing Company

PREPARATORY to building in the river channel the last section of the Brunet Falls dam, described in Engineering Record, April 5, 1913, page 368, provisions were made for passing the water of the Chippewa River through six conduits or tunnels built in the foundations of the power house. It was decided just before the water was turned from the river channel that four tunnels would take care of the season's flow, as the runoff proved to be much smaller during the winter of 1911-12 than had been expected. Forms were then quickly built and the concrete poured, the draft tubes being formed at the same time in the lower end of the two tunnel sections closed at this time.

The tunnels were 37 ft. long and 16 ft. square, with the exception that the roof was arched, the areas of cross-section being 243 sq. ft. The tunnels extended through the mass-work foundation under the flumes and discharged into the tailrace compartment under the power house. The division walls between tunnels, which supported the building and flumes, were 5 ft. thick, except at keyways, where they were 3 ft. thick, as were also the walls under the power house. In closing the last four tunnels only an 8-ft. concrete wall was built across the head end, thus leaving the filling of the greater part of the tunnel together with the building of the draft tubes to be done at some later date.


El. 123.0
Temporary Wooden Fill in all Keys
To be filled in at future date
4" Pipe
4'6"
Bags of Concrete

Discharge Opening Through Power House

KEYWAYS

When the tunnels were first built, keyways, 1 ft. deep and 4½ ft. wide, were carried around the openings in side walls, floor and arch, so as to lock in the new concrete work when the tunnels were closed. Near the head end of the tunnels in one of the keyways reinforcing bars with hook ends had been extended through the face of the concrete in both floor and arch to effect a positive connection with the bars in the closing mass of concrete. Two plans were suggested to effect the closure. The consulting engineers proposed to shut off the water by means of stop logs to slide in concrete grooves formed in slight projections on the upstream side of the dam. The second plan, not approved by the consulting engineers, was to use a flap gate hinged at the top to cover the upstream entrances of the tunnel. This latter method was followed and the difficulties encountered are herein set forth. In order to use 12 x 12-in. timbers for the gate, each tunnel was divided into two openings by a concrete post, 1½ ft. wide by 8 ft. deep. The timbers in these gates were held together by being bolted to three cross timbers. The entire gate was then hung from three extra heavy wrought-iron hinges embedded in the concrete above the openings. These gates were built in a horizontal position, swung up against the face of the concrete above the tunnels and held there while the water was discharging through the openings.

FLAP GATES FAIL TO SET

After completing the last section of the dam the cofferdams were opened and the river was allowed to discharge through five Coffin valves 8 ft. square built in tunnels located in the lower portion of the first five sections of the Tainter-gate dam. The flap gates were then lowered but did not set, owing to a large deposit of loose material which had been washed into the tunnel entrance. Two of the four gates had already been partly lowered, but these were raised to the surface of the water and the lower section, comprising four or five timbers of each of the gates, was cut loose and a second set of hinges placed to attach this portion to the main section so as to allow it to swing. The gates were again lowered, this time the main section swinging in against the concrete work, while the lower section rested on the deposit of loose material. These lower flaps of the gates slowly closed into place as the swifter water currents, under the partly closed gates, washed away the sand and gravel deposits.

Because of the fact that large boulders were caught between the lower flap of the second gate and the concrete work, it proved very difficult to close this particular gate. A clearance space had been allowed under the lower end of each of these gates, when closed, which allowed considerable water to pass through. To stop the leakage, first rock and then manure, cinders, sand, gravel and brush were placed on the upstream side. It proved very difficult properly to place these materials, as the headwater was eventually raised almost to full reservoir level and much of the material washed through into the compartment. The leaks were stopped sufficiently, however, to permit starting the pumps to empty the compartment.

DIVERS FOR TUNNEL 1

To shut out the tailwater from the compartment, stop-log timbers were placed in grooves provided for that purpose at the lower end of the compartment walls, which extended beyond the face of the building. Steam-driven centrifugal pumps had been set up, on the ends of these walls, to handle the water. Because of flood conditions the extremely high tailwater several times interrupted the work by rising above the stop-logs.

Tunnel 1 was the first to be closed. In the process of closing it the plan first pursued was to pump out the compartment, clean out the loose material, build forms and pour the concrete as mass work. However, after several unsuccessful attempts to empty the compartment, the pumps were shut down and a new method was pursued by having divers place the concrete under water. The material was mixed almost dry, placed in empty cement sacks and sent down to the divers, who built them up into a wall 8 ft. thick at the head end of the tunnel. At intervals drain pipes were placed in this wall so that the pressure between the timber flap gate and the new concrete wall might be relieved. After the wall had been built up to within 5 or 6 ft. of the roof the flow of water had been cut down to such an extent that it was possible to pump out the compartment, build forms and pour the remaining concrete as mass work. After allowing the concrete to set a sufficient length of time the pipes were capped.

DIFFERENT PROCEDURE FOR REMAINING TUNNELS

After tunnel 1 was finished the following method of closing the last three was adopted: Preliminary to the real work it was found necessary to add a carpet of brush and clay to the material which had already been placed outside the flap gate. This was for the purpose of sufficiently stopping the leakage so that the compartments might be emptied. Two pumps were used in each compartment, with the exception of the last one closed, where three were required. These included two 8-in. sand pumps and one 10-in. pump with smaller ports. After pumping out most of the water, enough loose material was removed so that a sheeting of 1-in. lumber could be laid against the inside face of the flap gates. This was done so as to control the leakage, which came in between the gate timbers in spurting streams, shooting all over the compartment. This acted as a protection for the workmen and also kept the water from washing over the concrete.

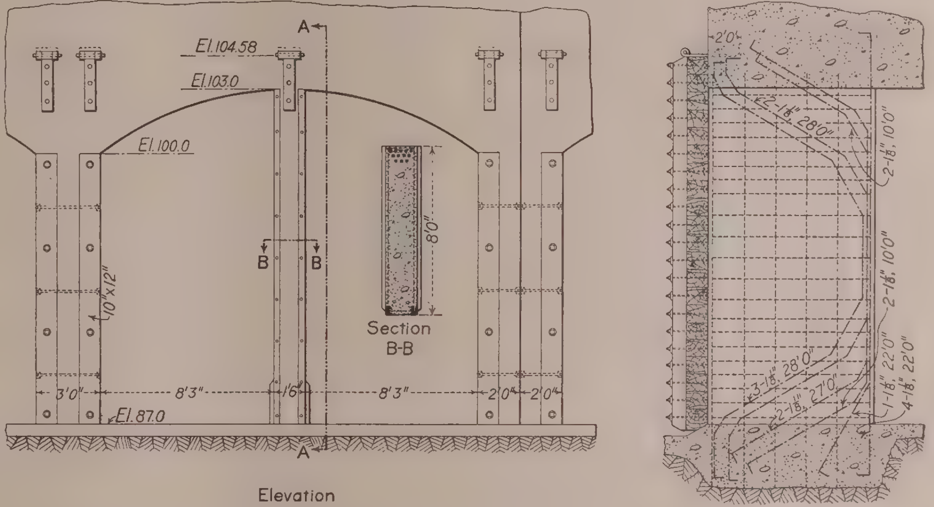
Following these preliminaries, 2 x 4-in. strips, 1 ft. apart, were first spiked firmly to the timbers of the gates and the sheeting was nailed to them. About 3 ft. from the floor a cross header of 2 x 4-in. lumber was spiked to the gate in such a way as to leave a clearance between the header and the vertical strips for the passage of the water from the various compartments between the strips. A 4-in. pipe was placed so as to conduct the accumulated water to a point beyond the outer form of the 8-ft. wall. This pipe was flanged so that it might secure a good grip on the sheeting and thus prevent its being blown out. Then after all sand, gravel, boulders and other material had been removed, all old form work was stripped from the keyways. These had been left in place, instead of removing them before the water was turned through the tunnels, so that a smooth surface would be presented to the running stream.

USE OF SACKS OF CONCRETE

After about 3 ft. of the outer wall form was built reinforcing bars, shown in the drawing, were placed by hooking them through the hook rods left in the old concrete. Back 3 in. from the bars, so as to give clearance for the mass concrete entirely to surround the steel, was placed a single row of sacks filled with concrete. These were built up to a height of 3 or 4 ft. and up against the sheeting spiked on the face of the gate. A good space was left between the sacks and the gate for a water pocket. From this pocket flanged pipes, of sufficient size and number to take care of the water, were laid through the row of sacks and extended beyond the forms at the lower end of the 8-ft. wall. In this manner it was possible to exclude all currents from the water inside the forms. This water was kept down to within 2 ft. of the floor, but could not be drawn down lower by the pumps without exposing the foot valves.

CONCRETE PLACED DRY

Dry concrete was deposited in this water through a spout so as to prevent washing out the cement. After the concrete had been built up above the sacks there was no more difficulty in placing. It could be poured in the usual manner, therefore, and by spouting it down from the top of the flumes it did not take long to fill the forms. After the concrete had been brought up to within 1 ft. of the roof it was allowed to stand over night for settlement, and the remaining material was then placed as grout through 12-in. tubes, placed specially for this work when the main structure was built. Vent pipes 3 in. in diameter were also placed at that time so as to allow the exit of all air, thus insuring that the keyways would be completely filled and sealed. With the grout pipes at one end and the vent pipes at the other end of the highest point in the keyway it was possible to secure the desired result. After allowing the concrete to secure a proper set the drain pipes were capped.



Elevation
Door not in Place

Section A-A

Details of Discharge Opening, Showing Arrangement of Doors and Piers

CLOSING LAST TUNNEL

In closing the last tunnel more difficulties were met. The leakage under the gate was especially hard to stop, and even after the pumps had drawn down the water, breaks occurred several times, necessitating shutting them down and depositing more material in the reservoir above the gate. By depositing the clay at different points it was found that water found its way through the lower layer of washed gravel and boulders, under the mass of placed material and under the gate. This necessitated making a very extensive clay apron.

Even after the water in the compartment had been drawn down to within 2 or 3 ft. of the floor, it was impossible to go on with the work until the opening under the gate had been partly closed, first by sticking small blocks of wood through the incoming water and across the opening at the outside of the gate. To place these blocks it was necessary for the men to lie down in the water with only their faces above the surface, and not altogether free at that from the streams from above. The water was also very cold, coming in as it did from the bottom of the reservoir. By using the above mentioned blocks the larger openings were closed, and after this was done a double thickness of 3-in. plank, fitted with splines, was driven down back of two sets of timbers which extended from the side walls to the center post, with knee braces to support the centers of the timbers. Both timbers and braces were pocketed into the concrete for proper support. The span from post to wall was 7 ft. 3 in. In this manner the heaviest flow of water was stopped, the remainder being conducted through drain pipes, and instead of three pumps being required, as at first, there was hardly enough leakage coming through to hold the priming in one pump.

As mentioned above, only an 8-ft. wall was built across the head end of the tunnel. It will not, however, be difficult to place the remaining concrete when it is desired to build the four draft tubes in these compartments. The upstream end is effectually sealed and the stop logs were left in place at the tailrace end. Hence it will not be difficult to pump out the compartment, clean out the keyways, build forms and place the concrete.

A DIFFERENCE IN WATER DENSITY at the lower ends of the Miraflores and Gatun locks at Panama has, according to the "Canal Record," developed a current of probably 3 or 4 miles per hour under certain conditions. At the stage of a downward lockage when the surface of water in the lower chamber has been brought down to evenness with that in the approach the water within the chamber is more than half fresh. It has, in consequence, less density than the sea water beyond the lower gates, and when the gates are opened the heavier water thrusts its way inward against the lighter, causing a current opposed to the outward passage of a vessel. The current being temporary, causes no serious inconvenience, and is being studied principally to determine accurately the conditions for the manipulation of the towing locomotives.

Girderless Reinforced Concrete Slabs

Proposed City Ordinance to Regulate the Design and Construction of Buildings of This Type and Discussion of Chicago Conditions

By ERNEST McCULLOUGH
M. Am. Soc C. E.; Consulting Engineer, Chicago

THE demand on the part of owners of buildings for ceilings without exposed beams and girders is being met by contractors who supply floor and roof slabs of reinforced concrete in which plate action is presumed to take place. The pressure now exerted on city officials to prepare design standards for this type of construction is considerable, especially since a number of men are designing structures in which girderless slabs are used without, according to the statements of the designers, conflicting with any patents.

Were it not for the fact that a number of patents have been granted, the girderless slab would have supplanted the usual beam and slab construction in a large per cent of the buildings erected within the past three years. In the city of Chicago the difficulty has been partially met by having each system considered alone, the design standards accepted by the building commissioner applying to the one system specifically mentioned in the documents relating to the standards. At the present time, however, there is an insistent demand that general design standards be adopted for the guidance of all designers, leaving the question of patent rights to be fought out after the design has been passed by the city's engineers.

CHICAGO METHODS

The procedure in Chicago has been as follows: A company secured a contract for a building in which a certain system of girderless slab was to be used. The owners of the patents thereupon applied to the building commissioner for the formulation of design standards to comply with the general building regulations. The building commissioner gave to the applicants a list of engineers with directions to strike out from that list the names of any engineers to whom the patentee might object. From the list of engineers not objected to the building commissioner appointed three to act as a board of examiners of the system and to prepare design standards. The entire expenses of the board were in all cases met by the owner of the patents, the city not being involved in any way financially.

The board, after appointment, held meetings at which all evidence possible to procure was presented and then submitted to the owners of the patents tentative design standards by the use of which a building was designed. Sixty days after a floor was poured a test load equal to twice the design load was placed on several adjacent panels and the deformations in the steel and concrete were obtained. With this data before them the Board of Examiners finally evolved design standards which were filed with the Building Department as documents not for public use and the system could thereafter be used in Chicago on precisely the same terms as common methods of design. The use of the system, however, is expressly reserved to the owners of the patents and no designs prepared by outsiders and in apparent conflict with these accepted standards of design will be passed upon at the City Hall.

The following systems have been approved in Chicago:

Cantilever flat slab—Concrete Steel Products Company, owner: A four-way system of reinforcement resembling in appearance to the casual observer the Turner mushroom system. The report on this system was dated Oct. 21, 1911, and the board of examiners consisted of A. N. Talbot, R. M. Shankland and L. E. Ritter.

Akme system—The Condron Company, owner: A two-way system based on wide shallow girders over the columns carrying an interior rectangular panel. The report on this system was dated May 29, 1912, and the board of examiners consisted of A. N. Talbot, W. K. Hatt and L. E. Ritter.

Corr-plate floor—The Corrugated Bar Company, owner: A two-way system in which the reinforcement is placed in narrow strips parallel with the sides and ends of the panels. The report on this system was dated Dec. 28, 1912, and the board of examiners consisted of A. N. Talbot, W. K. Hatt and Ernest McCullough.

The two systems following have had buildings erected on the tentative design standards; tests were made but the reports are not quite ready, the members of the boards being still at work on the data collected:

Mushroom system—C. A. P. Turner, owner: A four-way system with light rods running over the heads of the columns in essentially a radial manner, with heavy hooped columns having projecting cantilever bars at the top into the slab and with circumferential bands around the head in the slab. The board of examiners consists of Professor Eddy, Professor Kavanaugh and F. A. Randall.

Barton spider web system—Francis M. Barton, owner: This is a four-way system in the bottom to care for positive moments and a two-way system over the column heads to care for negative moments. The head is a factory-made mat of rods shipped to the job, thus insuring the positive location of the steel at this important point. The bottom steel is not bent and sometimes the depressed drop is omitted, the steel being relied upon in the bottom to assist in taking care of compression over the head. The board of examiners consists of A. N. Talbot, W. K. Hatt and Ernest McCullough.

UNPATENTED SYSTEMS

The foregoing systems are all patented. A number of well-known engineers and firms of contractors are exploiting girderless slab construction on which no patents have been asked, but when they attempt to take contracts in Chicago they find themselves balked by the present method for taking care of this matter. The cost of a commission, together with the tests, is not far from $3,000, and men who are not protected by patents hesitate to go to this expense in order to secure perhaps only one contract, after which their method will be open to the use of all competent designers.

One company, however, having a system not yet patented, has applied for a board of examiners; the tentative design standards have been drawn, but the test has not yet been made. The company mentioned is the Trussed Concrete Steel Company of Detroit. The system, called the flat ceiling construction, some features of which may be patented, is a four-way system closely resembling in appearance to the casual observer both the cantilever flat slab and the mushroom system. The board of examiners consists of A. N. Talbot, W. K. Hatt and Ernest McCullough.

ORDINANCE NEEDED

The writer has had occasion to go into this matter pretty deeply, and has closely followed the many discussions relating to the design of girderless slabs. He has been asked by several engineers connected with city building departments to submit to them an outline of a general ordinance that will apply to all the systems now in use and such others patterned after them as may appear. Such an ordinance is not easy to prepare. Theoretical discussions seem to have led us to no place in particular. Men having the courage of their convictions, based on the small support afforded them by Grashof's empirical formulas for thin plates supported on a number of points, have been responsible for the present knowledge we have of how to construct such floors, able to carry the loads for which they are designed. Mr. Turner deserves well of the profession, for his pioneer work has made the girderless slab floor something to reckon with. The writer, however, is not prepared to go the full way with Mr. Turner, but this feeling is shared in common with many engineers. In the books by Taylor and Thompson and by Turneaure and Maurer methods are given for computing the stresses in such slabs, but they are apparently followed by few designers. Mr. Saurbrey, in "Reinforced Concrete Buildings," gives a method that may be said to be fairly consistent with methods followed by those who have evolved from "cut-and-try" designing rules practical structures. The greater number of men who are engaged in designing and erecting buildings with girderless plate floors and roofs started with the work done by Mr. Turner as a basis. An ordinance prepared for the general use of designers must for the present be a composite of the methods followed by designers of several kinds of such slabs, the differences being solely in the arrangement of the steel.

STRESSES

A careful study of published results of experiments has convinced the writer that the concrete does more than carry compressive stresses. While conservative design in reinforced concrete neglects all consideration of the tensional strength of the concrete, in plates it must be considered. It is barely possible that the earlier floors were probably made with a somewhat richer mix than is considered right for ordinary structural work, not of the sloppy consistency now generally used. Competition has led to a less rich and more poorly mixed concrete; this perhaps accounts for cracks not observed in the first floors put down. When the concrete is first class and the usual bending moments assumed for such slabs are used, it is apparently immaterial which way the steel is placed, provided there is enough. Unbalanced moments are thrown into the columns and future studies and tests must pay more attention to this effect upon columns. At present we seem to have good information as to the action of stresses in the plates.

DRAFT OF PROPOSED ORDINANCE

The writer submits the following as a tentative draft for an ordinance to regulate the design and construction of girderless slabs of reinforced concrete, intending to complete it and put it into as presentable shape as possible, after the readers of Engineering Record have finished discussing it. He hopes that when the discussion is completed it will in the main be recognizable. The coefficients given are those actually in use. The methods described are actually in use. The ordinance is supposed to be supplementary to ordinances already in existence, which shortens it because limitations as to stresses are fixed in the ordinances.

Definition.—Girderless slabs are slabs of reinforced concrete supported on columns, with or without enlarged heads; the whole construction being hingeless and without visible beams or girders, although the construction may be such as to admit of hollowed panelled spaces in the ceiling or admit of smooth ceilings with depressed panels in the floors for the purpose of reducing the dead weight.

Columns.—The supporting columns are considered to be an integral part of the construction and in their design the live load shall be reduced from floor to floor as permitted in the building ordinance. The wind effect, if any, on the columns shall be investigated and allowed for. To provide for unbalanced moments transmitted from the slabs the columns shall be designed to carry an additional direct load as follows, after reducing for live load and adding the wind load: For columns supporting the roof, 10 per cent; for the next floor, 15 per cent; for the next floor, 20 per cent, and for each floor following decrease this allowance by 5 per cent cumulatively on each floor until a total of 10 per cent is reached, which amount shall be considered on each floor to the footings. This additional load shall not be considered as part of the load in designing footings. Columns shall preferably be of structural steel fireproofed, or shall be hooped columns designed by the formula

$$P = f_c\,(A_c + nA_c + 2.4nA_h)$$

in which P = load on column; f_c = unit compressive stress on concrete within the core; n = ratio of stress between steel and concrete; A_c = area of concrete within the core; A_s = area of vertical steel; A_h = hooping steel expressed in terms of cross-sectional area of column within the core.

The hooping shall be wound spirally round the core with a pitch not greater than one-sixth the core diameter for cores less than 18 in. in diameter; one-eighth the core diameter for cores more than 18 in. and not more than 24 in. in diameter; one-tenth the core diameter for cores more than 24 in. in diameter, provided that the maximum pitch shall not exceed 3 in. The core shall be measured to the inside of the hooping steel and not to the center of same. The hooping steel shall have a covering of not less than 1½ in. of concrete, measured from the outside of the steel. The vertical steel shall not exceed 6 per cent of the bulk of the material inclosed within the hooping, and not less than eight bars shall be used, spaced equally around the circumference with an interval between them, measured center to center of the bars, of not less than three times their thickness or diameter. The hooping steel shall be not less than one-half of 1 per cent of the bulk inclosed within the hooping and shall not exceed 1½ per cent. No concrete outside the hooping shall be considered in computing the strength of the column. The outside diameter of a column shall in no case be less than one-twelfth the panel length measured between centers of adjacent columns.

Column Capital—The column capital is hereby defined as the flared portion of the column at the upper end. The diameter of the column capital shall be measured where its vertical thickness is at least 1½ in., and this diameter shall be not less than 0.225 the panel length measured between centers of adjacent columns. The slope of the column capital shall nowhere make an angle with the vertical of more than 55 deg.

Column Drop—When the design provides within a square or rectangle around the column capital a greater thickness of slab than for the remainder of the panel, the least width of this drop shall be not less than four-tenths of the panel length. The depth of the panel drop shall be computed on the basis of furnishing the necessary compressive strength over the column and also providing for punching shear. The increased thickness, or depth, of the drop shall not be more than four-tenths of the minimum thickness of the slab, unless the increase in thickness be made in two or more steps or stages.

Slab Thickness—Provided the slab thickness in the center of the panel and at the middle of the side shall be not less than 1/32 of the panel length, the thickness shall be found by the formula,

$$t = \sqrt{0.0006\,wl^2}$$

in which t = thickness of slab in inches; w = unit total load per square foot; l = panel length in feet. The concrete protection under the steel in the bottom of the slab and over the steel in the top of the slab shall have a minimum thickness of 1 in., provided that it be not less than one-and-one-half the thickness or diameter of the rods or bars used, measured from the outside of same.

Shearing Stress—The shearing stress shall be calculated at critical sections for the punching shear and the limiting stress shall be 3/50 of the ultimate compressive strength of the concrete. In computing shear as diagonal tension the maximum shall not exceed the amount allowed for punching shear, one-third of which value may be carried by the concrete, the remainder to be taken care of by bent up bars or stirrups or by both.

Bending Moment Coefficients—The bending coefficient for negative moment over the columns shall be 1/15 and the bending coefficient for positive moment at mid-span on the sides of square panels shall be 1/25. The negative bending coefficient in the center of the sides shall be double the negative coefficient over the supports for steel in a parallel direction and the positive coefficient in the center of the panel shall be double the positive coefficient in the sides for steel in a parallel direction. If the width of column capital and column drop vary from the least dimensions herein specified the bending coefficients may be varied as follows: The sum of the positive coefficient and one end negative coefficient around the sides of the panel shall be 1/9. The positive coefficient in the middle of the panel length shall be 1/15 of the per cent the clear span between edges of adjacent column drops bears to the panel length between adjacent columns.

Point of Inflection—The point of inflection shall be considered as being one-fourth of the panel length out from the center of the columns, except when the width of column drop exceeds the minimum, when the point of inflection shall be located by the formula

$$R = L/2 \times (1 - \sqrt{9}/c)$$

in which L = panel length measured between centers of adjacent columns; R = distance from center of column to point of inflection; c = positive moment coefficient in side of panel at middle of span. One-third of the steel over the heads of columns shall extend to the point of inflection and there be bent down, and not less than one-third shall extend at least two-tenths of the panel length beyond the centers of the columns, and be bent down for anchorage at an angle with the horizontal of not more than forty-five degrees.

STEEL REINFORCEMENT

Placing of Steel—Spacers and supports to insure the positive location and position of reinforcement must be used. Steel passing from regions of negative stress to regions of positive stress, and vice versa, must be bent. The use of light rods or bars assumed to fall into location by sagging is prohibited. Rods or bars lapped at points of maximum stress shall be placed parallel to each other for a length of 1 ft., but separated by a space not less than twice the thickness or diameter, and beyond this lap of 1 ft. shall extend a length sufficient to develop bond, under the fiber stress used, bending gradually in the compressive area of the concrete. Slabs may be reinforced (1) with two bands of steel over the column heads parallel with the sides and two bands diagonally over the column heads, making a four-way reinforcement for positive and negative moments; with steel across the side bands to take care of negative moment at middle of spans; (2) with four-way reinforcement for positive moments and two-way reinforcement for negative moments, with steel across the sides for negative moment; (3) with two-way reinforcement along the sides of the panels and over the supports, carrying a rectangular interior panel the reinforcement of which shall go over the side bands for negative moment; (4) with two-way positive and negative reinforcement divided into strips having widths proportioned to the amount of bending moment carried by each strip. Steel in the bottom for positive reinforcement not bent to serve as negative reinforcement shall extend beyond the center line of columns a distance sufficient to develop bond but not less than forty diameters. Steel passing from regions of positive moment to regions of negative moment shall be bent up as follows: Not less than one-third at the point of inflection and not less than one-third at a point two-tenths the panel length from the centers of the columns, but computations shall be made to determine that enough steel is placed at regions of maximum shear to care for shearing stress. Steel placed across the side bands shall extend not less than forty diameters beyond the edge of a strip equal in width to the column capital. All steel shall be figured for bond and adhesion which shall be kept within the limits fixed by the building regulations.

Bending Moments of External Forces—

The bending moment for each panel for four-way reinforcement and combined four-way and two-way reinforcement equals wL^2/c, in which w = the sum of the live and dead loads per square foot; L, panel length measured between centers of adjacent columns; c, moment coefficient. The amount of steel available to resist the bending moment is to be taken as the total area of the steel in an angular distance around the column of 180 deg., this amount being equally distributed in square panels over the number of bands used, all the steel being assumed to be concentrated at the level of the center of gravity of the reinforcement. The width counted in any one band of reinforcement shall not be more than the diameter of the column capital plus five times the thickness of the floor and also not more than four-tenths of the panel length.

Compression in Concrete—The compressive stress at a section of the floor at column capitals shall be calculated to resist the negative bending moment herein specified on the basis of a width equal to the combined widths of the bands of steel over the head plus four times the depth of the slab multiplied by the number of said bands, provided the width shall not exceed one-half the panel width for each band. If a column drop is not used the effect of the compression steel near the bottom of the slab may be considered, provided it passes the center line of the column a distance of not less than forty diameters. To do this the areas of the rectangular bands shall be added to the rectangular components of the areas in the two diagonal bands.

Two-Way Panels.—For two-way reinforcement $M = wL^2/c$ for strips parallel with the sides, the coefficients on the sides and over the column heads for a width equal to one-fourth the panel width and for a strip one-half the panel width across the center parallel with the sides being those herein specified. Between the outside and the middle strips shall be two strips on both sides, each one-eighth the panel width, with coefficients equal, respectively, to five-sixths and four-sixths the coefficients for the middle strips. The compression in the concrete shall be computed on the basis of a width equal to the panel width for the bending moments across the entire panel.

When two-way slabs are designed on the assumption of wide shallow girders over the column heads carrying an interior panel the width of a girder shall be assumed as not more than one-half the panel width for the purpose of figuring compression in the concrete; it shall be assumed to carry half the total load consisting of the dead load plus eighty-five per cent of the live load on a panel. The reinforcing steel shall be placed in a width not greater than the width of the column cap, plus six times the thickness of the girder slab, provided that this does not exceed one-half the panel width.

The two-way reinforced suspended slab shall be considered to have dimensions greater than the clear spans between the edges of the girders by adding to the clear span twice the girder panel thickness. The bending moment on the span thus defined shall be $WL/12$, in which W shall be the proportion of load on the span, assuming that the distribution of load to each side is found by the formula $r = L^4/(L^4 + B^4)$, in which r = proportion of load carried by the transverse reinforcement; L = length of slab; B = breadth of slab. For a square slab, $r = 0.50$.

Walls and Openings—Girders or beams shall be constructed under walls and around openings and to carry concentrated loads, and the reinforcing rods and bars shall be anchored securely by bending or otherwise.

Wall Panels—The moment coefficients for wall panels shall be 20 per cent smaller than the coefficients for interior panels and sides of panels common to wall panels and interior panels shall have coefficients 10 per cent smaller than those for interior panels.

Rectangular Panels.—When the length of a panel does not exceed the breadth by more than 5 per cent, all computations shall be made on the basis of a square with sides equal to the mean of the length and breadth. In no rectangular panel shall the length exceed four-thirds the width.

For panels with length more than five per cent in excess of the breadth the slab shall first be designed for a bending moment based on a square panel whose diagonal is equal to the diagonal of the rectangular panel. The amount of steel in a band found by designing in this manner shall be that used in the diagonal bands. For cross bands the required amount of steel is to be found by multiplying the steel used in the design of the equivalent square panel, by the cube of the ratio found by dividing the length of the panel by the length of the side of the equivalent square panel for steel in the long direction, and by the cube of the ratio found by dividing the breadth of the panel by the length of the side of the equivalent square panel for steel in the short direction. The compressive stresses shall be calculated on the assumptions used in the calculation of compressive stresses in square panels.

Test of Workmanship—All deflections under test load shall be taken at the center of the slab and shall be from the normal unloaded position of the slab. Two adjacent panels shall be uniformly loaded over their entire area with a load equal to the dead load plus twice the live load, thus obtaining twice the total design load, the dead weight of the construction being considered as part of the test load. The load shall remain in place for 24 hours. If the deflection under the test load does not exceed one-eight-hundredth of the span the slab will be placarded to carry the full design live load. If it exceeds this amount of deflection and recovers not less than 80 per cent of the deflection within seven days after the load is removed, the slab will be placarded to carry the full design live load. If the deflection exceeds the allowable amount above specified and the recovery is less than 80 per cent in seven days after the removal of the test load, other tests shall be made on the same and on other panels, the results of which will determine the amount of live load the slabs will be permitted to carry.

A PRECIPITATION MAP of British Columbia which appears in a report recently issued by the water rights branch of the provincial Department of Lands shows by means of lines similar to contours the distribution of rainfall over the entire province for the year 1913. The maximum precipitation shown is 170 in. at Swanson Bay, while the minimum is 5 in., which was recorded at Ashcroft.

Special Representations in Specifications Control General Cautionary Clauses

Recent Decision of the United States Supreme Court, Reversing the Court of Claims

By WILLIAM B. KING
Of the Bar of Court of Claims, Washington, D. C.

THE opinion of the Supreme Court of the United States has just been printed in the case of Hollerbach & May vs. United States, an appeal from the Court of Claims. This reverses the decision of the Court of Claims, which was against the contractors, and directs judgment to be entered in their favor.

The suit was based upon a contract with the United States for the repair of Dam 1, Green River, Kentucky. The specifications contained the following general provisions, which appear frequently in engineering contracts of the United States, requiring contractors to take the risk of local conditions in making their bids:

"20. It is understood and agreed that the quantities given are approximate only, and that no claim shall be made against the United States on account of any excess or deficiency, absolute or relative, in the same. Bidders, or their authorized agents, are expected to examine the maps and drawings in this office, which are open to their inspection, to visit the locality of the work, and to make their own estimates of the facilities and difficulties attending the execution of the proposed contract, including local conditions, uncertainty of weather, and all other contingencies."

"70. Investigation. It is expected that each bidder will visit the site of this work, the office of the lock master, and the office of the local engineer and ascertain the nature of the work, the general character of the river as to floods and low water, and obtain the information necessary to enable him to make an intelligent proposal."

One paragraph of the specifications contained the following specific statement of exact conditions to be found behind the dam: "The dam is now backed for about 50 ft. with broken stone, sawdust, and sediment to a height of within 2 or 3 ft. of the crest, and it is expected that a cofferdam can be constructed with this stone, after which it can be backed with sawdust or other material."

MATERIAL BEHIND DAM

The Court of Claims found as a fact that, when the contractors proceeded with the work of removing the material behind the dam, they did not find "broken stone, sawdust and sediment," as stated in the above paragraph, but they found a "soft, slushy sediment" to an average depth of about 7 ft. and, below that to the bottom of the excavation, they found cribwork consisting of sound logs filled with stones.

The Court of Claims found that the additional cost to the contractors, over what the cost would have been had the conditions been as represented in the above paragraph, was $6,549.23. The Court of Claims said that this paragraph was "a positive and material representation as to a condition presumably within the knowledge of the Government, and upon which, in the absence of any other provision or warranty, the plaintiffs had a right to rely."

In spite of this, that court held that the cautionary provisions of paragraphs 20 and 70, as quoted, prevented the representation from being a warranty and took away any right which the contractors had to rely upon it.

The Supreme Court considered the decision in the New York drydock case of Simpson vs. United States, upon which the Court of Claims relied, and said that it had not sufficient similarity to this case to form a precedent for the decision of the Court of Claims. It then contrasted the general provisions of paragraphs 20 and 70 with the particularity of the description of the old dam, as above quoted, and the conditions to be found behind it. "Obviously," the Supreme Court said, "this made it much more expensive to do the work than if the representation inserted by the Government in the specifications of its own preparation had been true and only the character of material had been found which the specification unequivocally asserted was there."

The result is stated in the concluding paragraph of the opinion, as follows: "A government contract should be interpreted as are contracts between individuals, with a view to ascertaining the intention of the parties and to give it effect accordingly, if that can be done consistently with the terms of the instrument. In paragraph 33 the specifications spoke with certainty as to a part of the conditions to be encountered by the claimants. True, the claimants might have penetrated the 7 ft. of soft slushy sediment by means which would have discovered the long crib work filled with stones which was concealed below, but the specifications assured them of the character of the material, a matter concerning which the Government might be presumed to speak with knowledge and authority. We think this positive statement of the specifications must be taken as true and binding upon the Government and that upon it, rather than upon the claimants, must fall the loss resulting from such mistaken representations. We think it would be going quite too far to interpret the general language of the other paragraphs as requiring independent investigation of facts which the specifications furnished by the Government as a basis of the contract left in no doubt. If the Government wished to leave the matter open to the independent investigation of the claimants it might easily have omitted the specification as to the character of the filling back of the dam. In its positive assertion of the nature of this much of the work it made a representation upon which the claimants had a right to rely without an investigation to prove its falsity. See United States vs. Stage Company, 199 U. S. 414, 424."

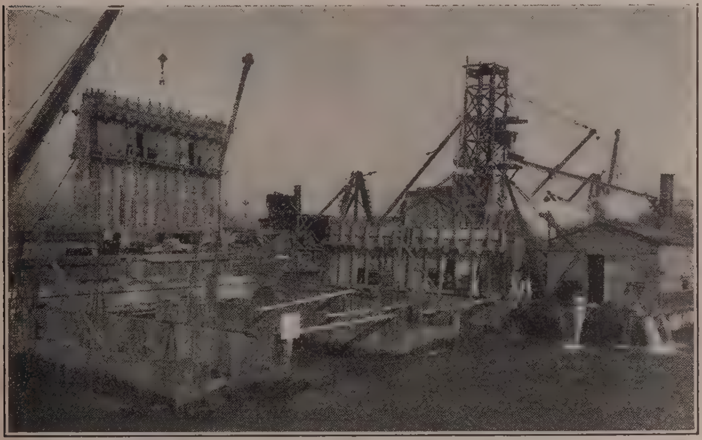
Accordingly, the judgment of the Court of Claims was reversed and directions were given to that court to enter judgment for the damages incurred because of the different character of material found behind the dam.

This decision is one of very general importance in the construction of contracts. In very many instances these contain some such general warning, coupled with particular statements of fact. These particular statements, it is now held, are binding upon the owner. The contractor has a right to rely upon them, notwithstanding the general warning clauses. It must be borne in mind, however, that this does not extend to cases where the particular statements are qualified by saying that given conditions may be expected. It only applies to cases where the conditions are stated as positive and definite matters of fact.

Construction of Gunpowder River Bridge

Nearly a Mile of Reinforced Concrete Flat Slab Spans Built on Piers with Foundations in Shallow Water and Deep Mud

THE Gunpowder River double-track bridge carries the main line of the Philadelphia, Baltimore & Washington branch of the Pennsylvania Railroad across an arm of the Chesapeake Bay about 18 miles north of Baltimore. It illustrates the most advanced type of slab-span bridges developed by the Pennsylvania Railroad Company and described in the Engineering Record of Aug. 9, 1913, page 144. The simple and comprehensive methods of construction here described were thoroughly planned and detailed in advance; with them, excellent results were obtained by an organization and equipment containing some special plant and providing for uninterrupted simultaneous progress on different classes of work.



Handling Complete Forms for Bush River Piers

Dredging, Formwork and Concreting for Piers

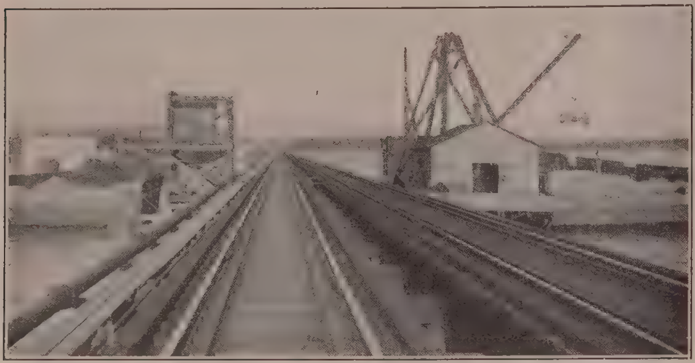
The bridge is 4961 ft. 10 in. long over all and carries two tracks about 18 ft. above low-water level at a point where there is an average tide of about 1½ ft. The water is generally 3 or 4 ft. deep. The shallow water caused serious difficulty, expense and delay in the construction operations because it limited the draft of the scows and interfered with the delivery of materials. The river bottom consists of 10 to 30 ft. of very soft mud, beyond which is harder mud, clay, sand, and gravel—the latter from 40 to 70 ft. below the surface—providing a hard and satisfactory foundation stratum.

The bridge has 191 reinforced-concrete piers about 24 ft. in total height, of which 186 are duplicates except for slight variations in the height. These have shafts 2 ft. 8 in. x 28 ft. 4 in. on top and from 15 to 19 ft. high, with a 10 x 35-ft. footing cast around the tops of forty-three wooden piles, from 30 to 70 ft. long, driven to refusal. The other piers are similar, varying slightly in dimensions. The piers support two short bascule plate-girder spans, four special short spans, and the 184 regular spans 26 ft. long, all of them, excepting the plate-girder spans, being flat reinforced concrete slabs, 34 in. thick and 28 ft. 4 in. wide, exclusive of the curb which increases the width of the upper part to 30 ft. 8 in.

In accordance with the indications of the test borings made about 300 ft. apart, the foundation piles were ordered with lengths varying from 30 to 70 ft. to provide for a penetration of several feet in the hard underlying gravel and clay stratum. They were driven by Arnott and Vulcan steam hammers at a maximum rate of sixty 65-ft. piles in ten hours by one hammer. The hammers were provided with a McDermott patent base and a cast-iron follower which drove the piles to an average depth of 3 ft. below the surface of the mud without much battering. Of the forty-three piles in each regular pier, twelve were battered 6 in. longitudinally or transversely. These piles were driven by a special machine which drove all of those battered in one direction for several piers, and then returned driving those battered in the opposite direction, and so on.

COFFERDAMS

After all of the piles for each pier were driven, they were enclosed by rectangular sheet-pile cofferdams nominally 10 ft. wide and 35 ft. long inside. The 6 x 10 or 12-in. sheet-piles, 24 ft. long, had dovetailed spruce splines spiked to them, and



Unloading Aggregate by Link-Belt Conveyor under Tracks

Driving Sheet-Pile Cofferdams for Gunpowder River Bridge

were driven between single inside and outside rows of 10 x 10-in. waling pieces just above water level. They were driven at the rate of 100 piles or one complete cofferdam per day by a No. 4 Arnott steam hammer suspended from the 50-ft. boom of a derrick mounted on a barge. The tops were driven to within about 4 ft. of water level, thus providing a penetration of 10 to 14 ft. below the bottom of the concrete footing.

The original specifications provided for a continuous trench 4 ft. deep with sloping sides and a bottom as wide as the length of the pier, to be dredged across the river before the foundation piles were driven. This involved a minimum excavation of about 63,000 yd. of original dredging, besides what would have necessarily been repeated on account of filling in, but it was finally determined to omit dredging except under the piers.

One sheet-pile was omitted in each cofferdam to provide a gate through which the river water entered freely and afforded abundant supply for the suction of an 8-in. centrifugal pump installed on a barge and delivering through a maximum length of 2850 ft. of 10-in. spiral-riveted 16-gage galvanized-steel discharge pipe. This pump easily dredged the mud in the cofferdam to the specified elevation, and deposited it behind a sheet-pile cofferdam 200 ft. long, closed at one end, which, according to government requirements, was built along the river bank to prevent the mud from obstructing the channel. The density of the mud was considerably increased by the displacement and impact of driving, and the material was successfully cut out by a 2-in. hydraulic jet under 200 lb. pressure. The excavation was carried down about 6 ft. below the original surface of the bottom by two dredge boats, each of which usually finished one cofferdam per day. The total amount of dredging was about 16,000 yd.

CONCRETING PIER FOOTINGS UNDER WATER

For most of the piers the concrete was mixed in a floating plant and delivered in 1-yd. bottom-dump buckets, lowered below the surface of the water by derricks, and emptied at the proper elevation by operating the flaps with latch strings. Nominally each footing required about 78 yd. of concrete filling the entire cofferdam to a height of about 6 ft. The surface of the concrete was carefully sounded with rods and it was deposited so accurately that when the cofferdam was unwatered the top, from 1 to 4 ft. below water level, was usually nearly horizontal.

At first, when the concrete was deposited directly on the soft mud, it was found that from 6 to 24 in. of laitance and mud accumulated on it and required considerable time and labor for its removal before the pier construction could be continued. This trouble was largely obviated by dumping 18 in. of gravel in the cofferdam between the pile tops before the concrete was deposited. The gravel was covered with a 6-in. layer of 1:3 cement mortar which was allowed to set 24 hours before the concrete was deposited on it. Very little mud or laitance appeared on top of this mortar, so that after it had been examined a few times it was considered unnecessary to clear it off before placing the foundation concrete. After the cofferdam was pumped out, and the laitance, about 6 in. thick, cleaned off, the forms were built for the pedestals, which, together with the shafts, were concreted in the dry. In the worst place, where no gravel seal was used before the concrete was deposited, there was found 24 in. of mud and laitance on top of the foundation concrete, but test borings made in the latter showed it to be perfectly good and sound.

PIER CONSTRUCTION

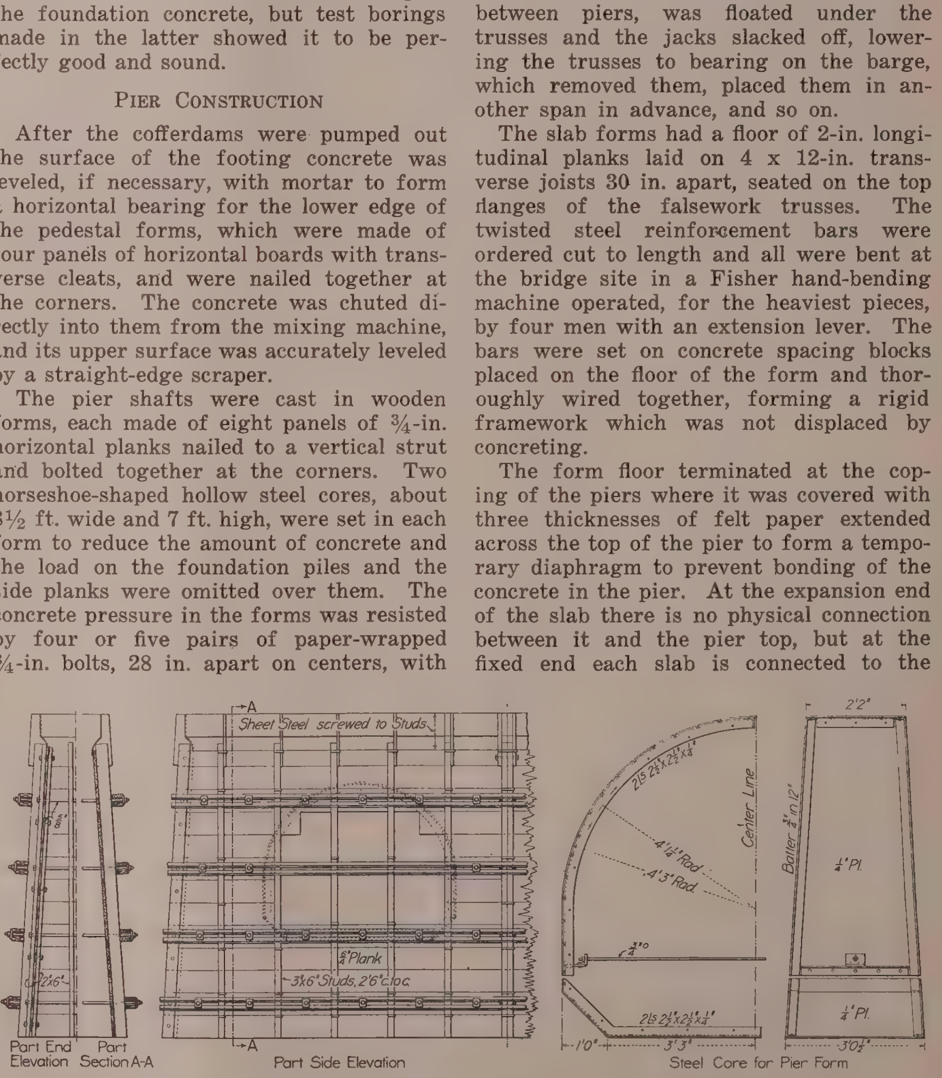
After the cofferdams were pumped out the surface of the footing concrete was leveled, if necessary, with mortar to form a horizontal bearing for the lower edge of the pedestal forms, which were made of four panels of horizontal boards with transverse cleats, and were nailed together at the corners. The concrete was chuted directly into them from the mixing machine, and its upper surface was accurately leveled by a straight-edge scraper.

The pier shafts were cast in wooden forms, each made of eight panels of ¾-in. horizontal planks nailed to a vertical strut and bolted together at the corners. Two horseshoe-shaped hollow steel cores, about 8½ ft. wide and 7 ft. high, were set in each form to reduce the amount of concrete and the load on the foundation piles and the side planks were omitted over them. The concrete pressure in the forms was resisted by four or five pairs of paper-wrapped ¾-in. bolts, 28 in. apart on centers, with nut and washer bearings on pairs of 2 x 8-in. waling pieces. The panels were stripped when the concrete was eight days old and were used over and over again on successive piers. The ¾-in. tie-bolts were driven out by a backing tool and used over and over again, the threads being recut when necessary.

Each of the regular-span slabs contains about 14 tons of reinforcement steel and has a total weight of 160 tons. They were cast in forms, each of which was supported on five light, riveted steel falsework trusses 22½ ft. long and 5 ft. deep on centers. The trusses were blocked and wedged up on the tops of wooden bents seated on the offset footings of the piers. They were adjusted to exact level by jackscrews and after the slab concrete was 15 days old a wooden barge, built to fit exactly the space between piers, was floated under the trusses and the jacks slacked off, lowering the trusses to bearing on the barge, which removed them, placed them in another span in advance, and so on.

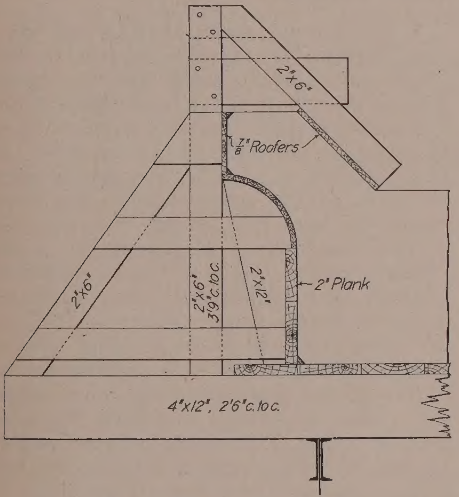
The slab forms had a floor of 2-in. longitudinal planks laid on 4 x 12-in. transverse joists 30 in. apart, seated on the top flanges of the falsework trusses. The twisted steel reinforcement bars were ordered cut to length and all were bent at the bridge site in a Fisher hand-bending machine operated, for the heaviest pieces, by four men with an extension lever. The bars were set on concrete spacing blocks placed on the floor of the form and thoroughly wired together, forming a rigid framework which was not displaced by concreting.

The form floor terminated at the coping of the piers where it was covered with three thicknesses of felt paper extended across the top of the pier to form a temporary diaphragm to prevent bonding of the concrete in the pier. At the expansion end of the slab there is no physical connection between it and the pier top, but at the fixed end each slab is connected to the



Wooden-Panel Form and Steel Core for Regular Pier Shaft pier by vertical steel dowels cast in the pier masonry and projecting nearly through the slabs. The slab concrete was delivered to the forms through a chute from the mixing plant at a maximum speed of about 1 yd. in 1½ minutes. A record of concreting four slabs in one day was made. The tops of the slabs were leveled off and then troweled to give a slight inclination toward each of the four drainage holes in every slab. With this exception no hand work or finishing was performed on any part of the concrete structure. Slabs were concreted alternately to save bulkheading between them. Bulkheads and side forms could be removed in twenty-four hours, and the floor forms removed after concrete has set from nine to fifteen days, according to weather. To prevent continuity between the ends of slabs two layers of deadening felt were placed at the end of completed slabs except at expansion joints, where eight or nine layers were used to give the required 1-in. joint.

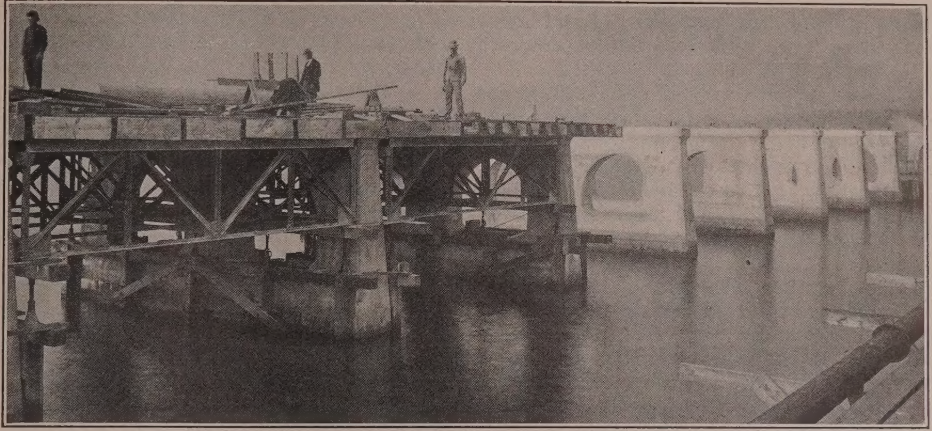
A small proportion of the sand and gravel for concrete was delivered by rail to insure an emergency supply, but the bulk of it was received on barges. As the water was too shallow to permit these to be towed directly to the site, they were moored adjacent to the old bridge, where they were unloaded by a derrick-boat with a clam-shell bucket delivering to an inclined Link Belt conveyor 16 in. wide and 50 ft. long, which passed through the railroad trestle under the track and discharged over a light-draft scow which was towed to the concrete plant.



Form for Roadway Slab

The concrete boat was equipped with a 1-yd. Smith mixing machine, a derrick with a 50-ft. boom, and a hoisting tower 40 ft. high. A clam-shell bucket unloaded sand and gravel from barges moored on both sides and delivered to elevated hopper-bottom storage bins supplying the mixing machines through a measuring box. The mixing machine discharged into a Ransome bucket, hoisted by a special engine to the top of the tower and there automatically discharged into a steel chute suspended from a light boom so as to command the entire span to be concreted. An average of 275 yd. of concrete per day was laid, the record being 360 yd. in 11 hours and 340 yd. in 10 hours.

The tower was provided with two hoppers, one of them 32 ft. above the deck to serve the pier forms. The other, which eventually was not used, was placed 8 ft. above the deck to serve the pedestal forms.



Steel Falsework Trusses for Gunpowder River Bridge Floor Slabs

Great care was taken to insure exact compliance with the specifications and exact proportions of aggregate and cement, the mixing and placing of concrete being constantly inspected by three assistants.

SHALLOW WATER PLANT

At the north end of the bridge for 1200 ft. the normal depth of 2 or 2½ ft. of water was so much reduced by almost continuous north winds that great difficulty was experienced in moving the floating equipment. Two special 20 x 40 ft. scows were braced together and on them were installed three boilers, a derrick with 60-ft. boom, and the dredging outfit. Concrete was mixed in a stationary plant on shore and delivered in 1-yd. hopper-bottom buckets on a service track laid on stringers placed on the completed piers. Removable bents of falsework were shifted from pier to pier, and enabled the buckets to be delivered from the plant in advance of the pier work, where they were handled by the dredge-boat derrick. The same derrick was also used for driving and pulling the cofferdam sheeting. It commanded three piers at once, so that it was able to remain busy in one place for several days and needed only to be moved on high tide when opportunity afforded. With it twenty-two piers were built during the cold weather, but the corresponding slabs were not constructed at that time.

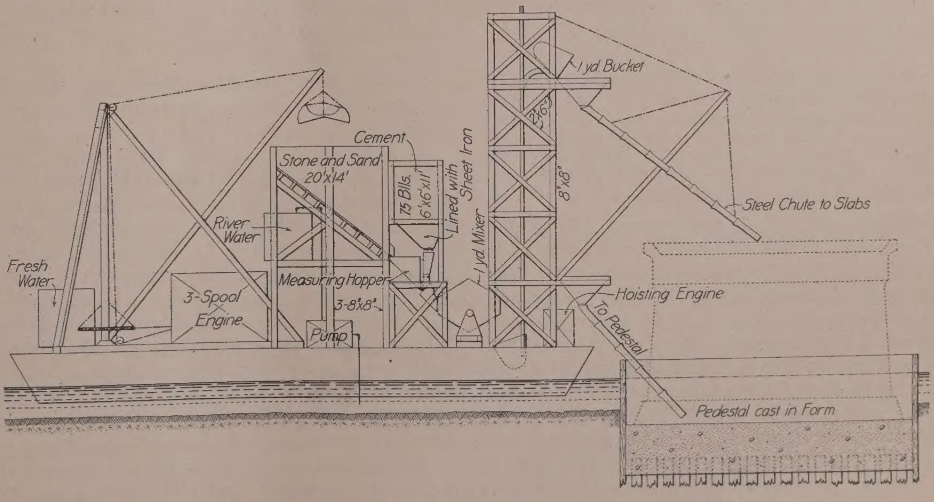
PRINCIPAL QUANTITIES

The most important items of the work included 16,000 yd. of river excavation dredged by the centrifugal pumps, 2275 yd. of rip-rap, 16,060 yd. of foundation concrete, 10,900 yd. of pier concrete, 14,440 yd. of slab concrete, 1000 yd. of extra foundation concrete, 8345 regular foundation piles, 800 extra piles, about 275,000 ft., board measure, of form lumber for piers and 70,000 ft. for slabs, about 275,000 ft. of 6 x 10-in. sheet piles and 100,000 ft. of 10 x 12-in. waling timbers, 230 tons of reinforcement steel for piers and 2480 tons for slabs. The sheet piles were driven over and over again, some of them as many as ten times, and were generally in good condition at the end of the job. The work was executed by an average force of about 250 men.

BUSH RIVER BRIDGE

The Bush River Bridge is similar to the Gunpowder River Bridge, except that its total length is only 2751 ft. 10 in. and that it has only 99 regular 26-ft. spans. The floating concrete plant was similar to that already described, except that it was installed on a 30 x 80-ft. barge, the width of which permitted the installation of a belt conveyor about 65 ft. long. The conveyor cantilevered 6 ft. beyond the stern of the boat and overhung the cement barge, allowing the bags of cement to be loaded there by one man and delivered to the platform of the concrete machine, thus dispensing with the services of several men employed in the other plant to deliver the concrete by hand.

The concrete was deposited directly on the mud bottom without the use of the gravel seal and only 6 or 8 in. of laitance formed. About 134 tons of reinforcement steel were required for the piers and 1260 tons for the slabs. This steel together with that required for the Gunpowder Bridge made a total of about 4000 tons, said to



Floating Mixer Plant Delivering Concrete through Chutes be, with the exception of the Panama Canal work, the largest single order of concrete reinforcement yet placed.

The work was designed and executed under the directon of the engineering department of the Pennsylvania Railroad, of which A. C. Shand is chief engineer. The design was personally directed by H. R. Leonard, engineer of bridges, and the execution was in charge of C. W. Thom, assistant engineer at the site. The contracts for both bridges together amounted to about $1,250,000 and were awarded to Henry Steers, Inc., New York, and Brann & Stuart, Philadelphia. F. H. Leonard and David Bonner were the assistant engineers in charge for the contractors of the Gunpowder and Bush River Bridges, respectively.

Book Reviews

The report of stream measurements in the Dominion of Canada for the year 1912 has just been issued. It is a volume of 450 pages, containing an introduction describing the work in general and then giving details of measurements in 33 drainage basins. In an appendix is a report on the rating of current meters by H. A. Brown, district hydrographer. (Ottawa, Department of the Interior.)

The sixth annual report of the Hydroelectric Power Commission of the Province of Ontario for year ended Oct. 31, 1913, has now appeared in its printed form. The book contains 322 pages and numerous photographs and diagrams. It is divided into five chapters, dealing with legal proceedings, transmission systems, operation of the systems, municipal work and hydraulic investigations. (Printed and Published by L. K. Cameron, Toronto.)

For a number of years the National Society for the Promotion of Industrial Education has been bringing together manufacturers and educators who are interested in vocational education. The seventh annual meeting was held in Grand Rapids, Mich., Oct. 23 to 25, 1913. The report of the proceedings makes a 6 x 9-in. volume of 261 pages, which has recently come from the press. Vocational education is discussed from many viewpoints, one of the principal addresses being by Mr. Redfield, Secretary of Commerce. There is considerable information regarding different systems of vocational education, particularly those enjoying public support. (New York, National Society for the Promotion of Industrial Education.)

HYDRAULICS. By Ernest H. Sprague, A. M. I. C. E., Assistant at University College, London; Lecturer at the Westminster Technical Institute; formerly Professor of Engineering at the Imperial Chinese Railway College, Shan-Hai-Kuan. Cloth, 4¾ x 7½ in.; 184 pages; 89 text figures. London, Scott, Greenwood & Son. 3½ shillings net.

This little book is volume 10 of the Broadway series of engineering handbooks issued by the English publisher. Its title is somewhat misleading in that it suggests a scope much wider than is covered in the book. The contents are arranged in five chapters entitled: Introduction and the principles of fluid pressure; liquids in motion; discharge through orifices, weirs, etc.; flow in pipes and channels, and pressure of water and application to motors and pumps. The chapters are brief but can be said to cover the subject well, although not exhaustively. Distributed throughout the book are 143 numerical examples, which greatly enhance its value. At the end of the book is a collection of useful data, four-place logarithmic tables, trigonometric functions, tables of squares and cubes, etc. The author states that the book has been compiled from his lecture notes and that it is aimed to be a text-book for students and engineers. In the opinion of the reviewer the book is well suited for its purpose.

DYNAMICS. By Horace Lamb, Sc.D., LL.D., F.R.S. Formerly Fellow of Trinity College, Cambridge, Professor of Mathematics in the Victoria University of Manchester. Cloth, 6 x 9 in.; 344 pages; 93 illustrations. London, Cambridge University Press. 10½ shillings. New York, G. P. Putnam's Sons. $4 net.

(Reviewed by Melvin L. Enger, Assistant Professor of Theoretical and Applied Mechanics, University of Illinois, Urbana.)

This book is a sequel to a treatise on statics published a little more than a year ago. The book forms an introduction to the dynamics of the astronomer and the mathematician, and is therefore not likely to be used in the engineering colleges of this country. The author has unusual ability in reducing demonstrations to their simplest terms, which, together with the historical sidelights with which the book abounds, should make it a very teachable book.

Many problems illustrating the principles are solved in the text, as for example, the complete solution of four problems by means of Lagrange's equations. At the end of each chapter is a collection of problems, nearly 350 in all. The author states in the preface that "preference has been given to examples which are simple rather than elaborate from the analytical point of view" and "which are really exercises in dynamical theory, and not merely algebraical or trigonometrical puzzles in disguise." In this he seems to have succeeded.

The typography, arrangement and figures are very good, but the index is very incomplete. The book is a distinct addition to the literature of this field.

Letters to the Editor

Design of Elevated Water Tanks

SIR: I desire to express my appreciation of the excellent article by Charles Pillsbury on the "Design of Elevated Water Tanks," which appeared in the Engineering Record of April 4, on page 390. I believe, however, that his comments on the relative advantages of the different forms of tank bottoms may give rise to some erroneous impressions.

When the conical or segmental bottom is joined to the sides with a suitable curve there is no difficulty in making a satisfactory and watertight connection. If improperly designed connections have resulted in failure, the same may be said of other engineering structures, and this should be taken as an argument for the proper design of connections rather than for the disapproval of any particular form.

I heartily agree with Mr. Pillsbury's comments on the hemispherical bottom, but do not concur with his statement that the elliptical bottom has the same advantages, since, with the proportions usually adopted, the tensile stresses in the lower portion of the latter are very much greater than those in the hemisphere, while the upper portion, near its connection to the sides, is subjected to compression.

GEORGE L. CHRISTY.

White Plains, N. Y

[Copy of Mr. Christy's letter was submitted to Mr. Pillsbury, whose reply follows.—EDITOR.]

SIR: In reply to Mr. Christy's letter, I should like to make the following suggestions:

1. Mr. Christy states that "when the conical or segmental bottom is joined to the side with a suitable curve there is no difficulty in making a satisfactory and watertight connection." This point is certainly well taken. If the connection is made as Mr. Christy suggests there is no structural objection to either type of bottom. Unfortunately a good many of the tanks with segmental and conical bottoms that have been built have had the connection made by flanging the top of the bottom plates. It is such a connection which is objectionable, as it is likely to leak and on a large tank is likely to lack the proper strength.

2. Mr. Christy states that in the elliptical bottom "the tensile stresses in the lower portion of the latter are very much greater than those in the hemisphere, while the upper portion, near its connection to the sides, is subjected to compression." It is a fact that as actually built the stresses in tanks of the same capacity are very nearly the same for either hemispherical or elliptical bottom. For instance, in the Chicago Bridge & Iron Works standard 100,000-gal. hemispherical-bottom tank the maximum stress in the bottom is 1100 lb. per linear inch and in the elliptical bottom of the same capacity 1300 lb. per linear inch. The compression occurs in the upper plates of an elliptical bottom only when the connecting radius is too short, and even so the amount of compression is insignificant except in tanks of very large capacity.

CHARLES S. PILLSBURY,
Engineer, Chicago Bridge & Iron Works.
Chicago.

Flood Prevention

SIR: The National and State legislative bodies have been requested by petition and otherwise to give relief by legislation to the flood-stricken districts of the United States, especially those affected by the high waters in 1913. The loss of life and property was so unusually large in March and April of last year that it might be classed as a providential calamity, but such interpretations would not satisfy the people of the devastated districts; they demanded a thorough investigation and report.

To comply with the requests made upon the National Government, a board of army engineers was appointed to make investigations and to report the principal causes for the flood and destruction of property. The cities of Columbus, Dayton and Hamilton, Ohio, and Indianapolis and several other cities along the Wabash and White Rivers in Indiana, employed eminent engineers to investigate and report causes of floods and to devise plans for prevention.

It has afforded me pleasure to read the different reports as published in the Engineering Record and other engineering journals, and to note that the principal cause, eliminating the factor of heavy rainfall, has been man himself, by encroaching upon the waterways with buildings, bridges and like structures.

The secret of flood prevention lies in removing all stream obstructions and constructing levees, diversion channels and reservoirs.

As an engineer of twenty-five years' practical experience in levee construction and contracting of water ways along the Wabash River, I want to ask, what are the principal factors used in ascertaining the additional height of water caused by stream contraction? I will confess that none of my past theoretical calculations have exactly conformed to my practical results, and as an engineer engaged in this class of work, I would welcome correspondence or a discussion of this subject.

J. S. SPIKER,
Vincennes, Ind. Consulting Engineer.

[Answers to Mr. Spiker's query are invited for publication in these columns.—EDITOR.]

An Earthquake-Proof Tower

SIR: My attention has been called to an article on the Sather campanile in your issue of March 14, page 312, which seems to give a slightly erroneous impression of the respective relations of Mr. Derleth and Mr. Cope to the work. For this uncertainty I presume I am partially responsible, as the article in question was, I believe, submitted to my office for correction before it was sent to your paper. Through an oversight the reference to the matter in question was not noted. In justice to all concerned I wish to state that Charles Derleth, Jr., is responsible for the civil-engineering work on this building, having been retained by me as engineer from the very beginning of the undertaking. The credit, therefore, like the responsibility, for the engineering work, which I consider of very exceptional quality, is due primarily to Professor Derleth. Mr. Cope was employed as assistant engineer under the direction of Professor Derleth and myself. Professor Derleth in the first instance laid out the scheme of calculation and drafting, directed and controlled it throughout and in the end completely checked the whole. His relation to the work was, therefore, something more than what is usually understood as that of consulting engineer. The design was worked up and the calculations made by Mr. Cope, and most satisfactorily, and it was not unnatural that he should have used the expression that the work was designed by him. I am writing, not to diminish the merit of Mr. Cope, who deserves high praise for his excellent work on the campanile, but to give the just measure of credit for the engineering design to Professor Derleth, to whom it rightfully belongs.

J. G. HOWARD,
Supervising Architect, University of California.
San Francisco.

Engineers' Fees

SIR: Your editorial in the Engineering Record of Feb. 7, page 150, regarding "Engineers' Fees," paves the way for much profitable discussion, which may lead to a partial solution of this vexing question. The complexity of business conditions in the several States, and even in different localities in the same States, makes any fixed rate impossible. The best that can be accomplished in rate fixing is to strike a general average, using ideal conditions as a basis to work from and establish a minimum rate, then let the engineer use his own judgment in making his charges.

In extensive undertakings subject to frequent interruption it would be next to impossible to state in advance what a reasonable fee would be. In such cases a per diem rate is the only fair way to settle the matter. An honest engineer working by the day can please any but an unreasonable client, and has an excellent opportunity to make a good reputation for himself. I know a successful builder who never bids on any work. He lives where competition is very keen and underbidding is frequent. His charges are so much per day plus a certain per cent of the total cost of the building. He is never out of work and enjoys an excellent reputation.

According to the letter of H. F. Tucker in the Engineering Record of Jan. 31 the engineers of the Pacific Northwest are "getting together" on the fee question. The spirit of the association is worthy of emulation. The rules and regulations of the California Association of Civil Engineers, although twenty-three years old, have suffered little change, and are still the basis of professional practice in California.

Santa Rosa, Cal. L. L. MILLS.

[Mr. Mills appends the regulations of the California association. Lack of space prevents their publication in detail.—EDITOR.]

Conditions of Railway Employment

SIR: In compliance with your editorial query in the issue of April 4, I am sending you herewith my ideas as to what is the matter with the railway-engineering departments. The whole trouble can be summed up in one word—"inefficiency." The average engineering department is not run strictly on a business basis, and one will never find on the same railroad the same methods used in different engineering offices.

The pay is low, it is true, but the work turned out neither in quality nor in quantity warrants increase in the pay of the men who do the work. The habit of hiring the first man who applies for a position and of paying practically no attention to his qualifications results naturally in forcing out the higher-grade men and filling their places with less efficient men. The work naturally suffers, and then there are complaints from the heads of departments about the inferiority of the work.

What is needed is, first and foremost, that the men and methods of doing work both in field and office be standardized, and thus a vast amount of waste time and red tape eliminated. This can be done by instituting some adaptation of civil-service examinations and by ranking the men according to their varying abilities, so determined. Then standard forms for field notes and office records should be issued and insisted upon, and field parties should work up their own notes instead of the usual method of sending them in to headquarters and having some poor devil spend several days trying to decipher hieroglyphics intelligible only to the man who wrote them.

With a proper system once firmly established and maintained I am fully convinced that a saving of 25 per cent could be attained, for I have made sufficient investigations along this line to be certain that the average field party, at any rate on maintenance work, spends from one-fifth to one-quarter of its time doing work that is, or should be, unnecessary. This saving of 25 per cent distributed among the remaining men in the form of increased salaries would stimulate them to better and more efficient work and would give them a pride in their occupation which at present is conspicuous by its absence.

In addition to the above, some kind of an organization among the rank and file themselves would work wonders. I mean something similar to the American Railway Association, holding periodic meetings at which the members could discuss the various phases and problems of their work. At present there is nothing to furnish the average man with new ideas, to keep him in touch with the work of others or to give him inspiration and, if need be, encouragement.

Everyone with whom I have talked on this subject agrees that at present there is great inefficiency in the department, the only problem being how best to eliminate it. The plan I have imperfectly outlined would at any rate be a move in the right direction.

Tacoma, Wash. D. L. SOLTAN.

Dustless Waterbound Macadam

SIR: As many are writing to me at different times concerning my new method of building waterbound macadam to make it dustless, I take this method of answering all. The method was first tried out by me in Columbus in 1907 and in the main consists simply of harrowing to the bottom the proper quantities of fines which when wet and rolled in the usual way will be forced up and between the stones to a height that will not cover the surface and yet will clinch the large spalls in a stable position or state of equilibrium. This may sound easier said than done, yet the harrowing is the secret of the whole thing. The resulting road has the appearance of a closely knit mosaic and with proper material and workmanship is long wearing and practically dustproof.

The too common mistake in building waterbound macadam is in trying to roll the fines into the interstices instead of rolling the large broken pieces into the grout. Such roads were nicknamed "upside down" roads, but I think the method is well understood now, as many highway engineers are specifying the same thing partly or wholly. I see many old worn-out or rather run-down stone pikes being rejuvenated at a small cost by using the scarifier and then working the fines down with a harrow. With a little new binder worked into these roads and the roads rerolled they can be made like new.

RAY S. BLINN,
City Engineer and Director of Public Service.
Mount Vernon, Ohio.

Reverting Specification for Paving Bitumens

SIR: There is considerable meat in the short article in your issue of April 4, page 393, reporting part of the address delivered by H. P. Pullar at the recent meeting of the Michigan Engineering Society. If specifications for bituminous pavements and highways are to be drawn so broadly that they will admit almost any type of bitumen, Mr. Pullar's suggestion is a good one. If specifications are classified it would be possible, as he suggests, to compel contractors to agree to use an asphalt complying with a specification of a particular class, and the State would thus be reasonably sure of getting a high-class material, if that was what it wanted and what it was paying for. With separate specifications for different classes of materials, each specification can be strictly drawn as Mr. Pullar points out, whereas with a blanket specification requirements are so broad that almost anything becomes acceptable. Naturally the award goes to the cheaper material.

The specification under which New York's highway department is now letting bituminous work is probably as bad an example as could be found of a wide-open specification. Under the heading of bituminous materials there are described three residual asphalts, one blown oil and one Bermudez asphalt. Any one of these materials is acceptable to the State, although hardly anybody would claim that they are all of equal quality. The result is, of course, that the contractor who bases his bid on materials of lowest quality, value and cost gets the contract and the State gets the cheapest kind of roads. Competition among the producers of the cheaper asphalts has reduced the price as low as 5½ cents a gallon and the State is going to get 5½-cent roads.

The policy suggested by Mr. Pullar is perhaps not an ideal one, but if adopted it would at least enable the State to select a bitumen desired for a particular piece of work, and to pay for the same in accordance with its relative value.

D. T. PIERCE,
Executive Assistant, Barber Asphalt Paving Company.
Philadelphia.

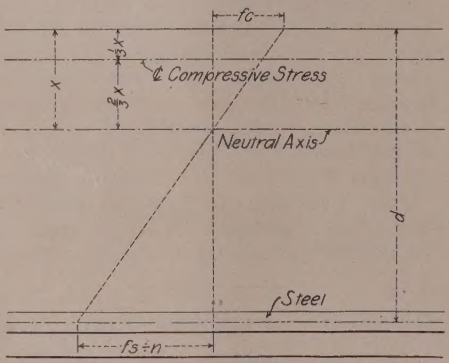
Design of Reinforced-Concrete Retaining Walls

SIR: Referring to the letter by Donald N. Becker appearing in your columns on page 86 of the issue of Jan. 17 under the title "Design of Reinforced-Concrete Retaining Walls," careful study of the design presented has failed to show me where the author has considered or provided for the direct compression produced in the concrete and steel, at the section of the wall considered, by its own dead weight. The author evidently intended to present a "method whereby correct results may be obtained" in designing the reinforcement for retaining walls where both bending and direct loads exist "without going through the laborious calculations" necessary in the exact method of solution of combined bending and direct-stress problems. However, his method deals only with the bending-moment stresses produced by the earth pressure and by eccentricity of the center of gravity of the wall with respect to the center of its base.

It is true that a slight saving in steel is effected by considering the negative bending moment produced by eccentricity of the center of gravity of the wall with respect to the center of its base when the front face of the wall has a greater batter than the rear face, as in the case which the author assumed; but when the front and rear faces have equal batter or are both vertical, no eccentricity of load, and hence no bending moment, results, and no saving of steel can be effected. In the case of walls with vertical front face and battered rear face—a not uncommon design for reinforced walls—a positive bending moment results which increases, instead of decreasing, the total bending moment. Additional steel must, therefore, be provided.

Mr. Becker's method, therefore, if used, would show a saving of steel in only few cases. It is hardly suitable for general use in retaining-wall design, on account of its greater length and limited application. Had he considered the direct compression resulting from the dead load of the wall, the extreme fiber stress in concrete at the section considered would have been increased by about 18 lb. per square inch.

Commenting on other features of the design, is it not a rather roundabout way to assume the dimensions of wall, and then by the "cut-and-try" method proportion the steel until the stresses in steel and concrete fall within the specified maximum values, when by the use of the regular formulæ for slab design, having the maximum values of f_s and f_c specified, one may obtain directly by one set of calculations the exact depth of slab and amount of steel necessary to keep the stresses in concrete and steel within the required limits for a given bending moment? From the straight-line theory of stress variation formulæ can be derived as follows: In the accompanying figure f_s = tensile stress in steel; f_c, compressive stress in concrete (extreme fiber); A_s, area of steel; M, bending moment in inch-pounds; b, width of beam; n, E_s/E_c. Then



$$(f_c/E_c):x = (f_s/E_s):(d-x)$$

or

$$(E_s/E_c):x = (f_s/f_c):(d-x)$$

But $E_s/E_c = n$. Therefore

$$n/x = f_s/f_c\,(d-x)$$

and

$$x = nf_cd/(f_s+nf_c) \qquad (1)$$

Now

$$M = (f_c \times bx/2)(d-x/3)$$

therefore

$$d = (2M + f_cbx^2/3)/f_cbx \qquad (2)$$

Substituting in equation (2) the value of x derived in equation (1),

$$d = \frac{2M + \frac{f_cb}{3}\left(\frac{dnf_c}{f_s+nf_c}\right)^2}{f_cb\left(\frac{dnf_c}{f_s+nf_c}\right)}$$

and

$$d^2 = \frac{2M}{\frac{f_c^2bn}{f_s+nf_c}\left[1-\frac{f_cn}{3\,(f_s+nf_c)}\right]}$$

Hence

$$d = \sqrt{\frac{2M}{\frac{f_c^2bn}{(f_s+nf_c)^2}\left[\frac{3f_s+2nf_c}{3}\right]}}$$

$$= \frac{f_s+nf_c}{f_c}\sqrt{\frac{6M}{bn\,(3f_s+2nf_c)}} \qquad (3)$$

Let the coefficient of $\sqrt{M}$ = the constant K. Then

$$K = \frac{f_s+nf_c}{f_c}\sqrt{\frac{6}{bn\,(3f_s+2nf_c)}} \qquad (4)$$

and

$$d = K\sqrt{M} \qquad (5)$$

Continuing,

$$A_sf_s = f_cbx/2$$

or

$$A_s = f_cbx/2f_s$$

Substituting the value of x from equation (1),

$$A_s = \frac{f_cb}{2f_s}\left(\frac{nf_cd}{f_s+nf_c}\right) = \frac{bnf_c^2d}{2f_s\,(f_s+nf_c)} \qquad (6)$$

Now let the coefficient of d = the constant K'. Then

$$K' = bnf_c^2/2f_c(f_s+nf_c) \qquad (7)$$

and

$$A_s = K'd \qquad (8)$$

The table gives the constants to be used in finding the values of d and A_s for various combinations of the values of f_c and f_s, and with b and n always constant and equal to 12 and 15 respectively.

CONSTANTS TO BE USED IN FORMULÆ

f_c	f_s	b	d	x	A_s
500	16,000	12	$0.034\sqrt{M}$	$0.32d$	$0.06d$
500	18,000	12	$0.035\sqrt{M}$	$0.29d$	$0.048d$
500	20,000	12	$0.037\sqrt{M}$	$0.27d$	$0.040d$
600	18,000	12	$0.0306\sqrt{M}$	$0.33d$	$0.067d$
600	16,000	12	$0.0297\sqrt{M}$	$0.36d$	$0.081d$
650	16,000	12	$0.0276\sqrt{M}$	$0.38d$	$0.088d$
650	18,000	12	$0.029\sqrt{M}$	$0.35d$	$0.075d$
650	20,000	12	$0.030\sqrt{M}$	$0.33d$	$0.060d$
700	18,000	12	$0.0273\sqrt{M}$	$0.37d$	$0.0863d$
750	16,000	12	$0.025\sqrt{M}$	$0.413d$	$0.116d$
750	18,000	12	$0.026\sqrt{M}$	$0.38d$	$0.096d$
750	20,000	12	$0.0266\sqrt{M}$	$0.36d$	$0.081d$
750	22,000	12	$0.0272\sqrt{M}$	$0.34d$	$0.070d$
750	24,000	12	$0.0285\sqrt{M}$	$0.32d$	$0.060d$
750	30,000	12	$0.030\sqrt{M}$	$0.272d$	$0.041d$
800	20,000	12	$0.025\sqrt{M}$	$0.37d$	$0.090d$
800	22,000	12	$0.026\sqrt{M}$	$0.35d$	$0.075d$
800	24,000	12	$0.026\sqrt{M}$	$0.33d$	$0.066d$
900	20,000	12	$0.023\sqrt{M}$	$0.40d$	$0.108d$
900	22,000	12	$0.024\sqrt{M}$	$0.38d$	$0.093d$
900	24,000	12	$0.024\sqrt{M}$	$0.36d$	$0.081d$
1000	20,000	12	$0.021\sqrt{M}$	$0.42d$	$0.126d$
1000	22,000	12	$0.022\sqrt{M}$	$0.40d$	$0.109d$
1000	24,000	12	$0.022\sqrt{M}$	$0.38d$	$0.095d$

The notation used is the same as that used in the report of the Joint Committee on Reinforced Concrete. Assuming f_s, 16,000 lb. per square inch, and f_c, 650 lb. per square inch, and substituting these values in the formulæ given, $d = 0.0276\sqrt{M}$ (M being in inch-pounds), $x = 0.38 \times d$ and $A_s = 0.088 \times d$. For $M = 517{,}250$ in.-lb., $d = 0.0276\sqrt{517{,}250} = 20$ in. and $A_s = 0.088 \times 20 = 1.75$ sq. in.

This design of course requires more steel than Mr. Becker's design, since the thickness of concrete is materially decreased. But if, for economical considerations, it is desirable to use more concrete and less steel, the value of f_c may then be assumed lower, say at 500 lb. per square inch, the value of f_s remaining at 16,000 lb. per square inch as a maximum. Then $d = 0.034\sqrt{M} = 24½$ in. and $A_s = 0.060\,d = 1.47$ sq. in.

EDGAR K. RUTH,
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Division of Structures.
Cincinnati.

GROSS OPERATING REVENUES for January on the large railroads of the United States were $1,004.91 per mile of road operated, as compared with $1,086.79 for January, 1913, and $931.55 for January, 1912. Operating expenses for the corresponding months were $776.32, $802.69 and $728.29 per mile, leaving $228.59, $284.10 and $203.26 respectively for net operating revenue per mile.